**BSC**

# Design Calculation or Analysis Cover Sheet

*Complete only applicable items.*

1. QA: QA
2. Page 1

| 3. System | 4. Document Identifier |
|---|---|
| Wet Handling Facility | 050-DBC-WH00-00100-000-00B |

5. Title
Wet Handling Facility (WHF) Concrete Slab and Diaphragm Design

6. Group
Civil/Structural/Architectural

7. Document Status Designation
☐ Preliminary ☒ Committed ☐ Confirmed ☐ Cancelled/Superseded

8. Notes/Comments

| Attachments | Total Number of Pages |
|---|---|
| ATTACHMENT A: WHF - Plans & Sections Sketches | 8 |
| ATTACHMENT B: WHF – Diaphragm Plans ( Showing Panel Design Cases ) | 3 |
| ATTACHMENT C: WHF – Diaphragm Panel Sketch ( Showing Chord Reinforcements ) | 2 |
| | |

**RECORD OF REVISIONS**

| 9. No. | 10. Reason For Revision | 11. Total # of Pgs. | 12. Last Pg. # | 13. Originator (Print/Sign/Date) | 14. Checker (Print/Sign/Date) | 15. EGS (Print/Sign/Date) | 16. Approved/Accepted (Print/Sign/Date) |
|---|---|---|---|---|---|---|---|
| 00A | Initial Issue | 76 | C2 | Pravin Udani | Ravinder Sanan | M. Ruben | Raj Rajagopal |
| 00B | Revised Pages 6 7, 9, 10 and 11. Revised (Mathcad) Pages 29, 30 and Pages 52 thru 55. This revision addresses CR Action # 11043-001 | ~~76~~ 75 PR 9/26/07 | C2 | Pravin Udani [signature] 9/19/07 | Ravinder Sanan [signature] 9/19/07 | M. Ruben [signature] 9/19/07 | Raj Rajagopal [signature] 9/19/07 |

## DISCLAIMER

The calculations contained in this document were developed by Bechtel SAIC Company, LLC (BSC) and are intended solely for the use of BSC in its work for the Yucca Mountain Project.

# CONTENTS

## ACRONYMS

| | |
|---|---|
| WHF | Wet Handling Facility |
| DL | Dead Load |
| LL | Live Load |
| C.G. | Center of Gravity |
| ITS | Important To Safety |
| HVAC | Heating, Ventilation and Air Conditioning |
| NRC | Nuclear Regulatory Commission |
| IBC | International Building Code |
| PDC | Project Design Criteria |
| TAD | Transport, Aging, and Disposal |
| YMP | Yucca Mountain Project |
| FE | Finite Element |
| FEs | Finite Elements |
| FEM | Finite Element Model |
| SASSI | System for Analysis of Soil-Structure Interaction |
| DBGM-2 | Design Basis Ground Motion (2000 Year Return Period) |
| SSI | Soil Structure Interaction |
| 3D | Three-Dimensional |
| BDBGM | Beyond Design Base Ground Motion (10000 Year Return Period) |

# 1. PURPOSE

The purpose of this calculation is to develop a preliminary design of the WHF concrete floor and roof slabs and diaphragms.

In this calculation, a representative sample of slabs will be designed. This sample includes the following cases: (See Attachment B, Sheet B2 - B3)

Locations of slabs/Diaphragms to be designed in this calc

| Cases | Elevation | Column lines | Thick | Mathcad Subscript | Design as |
|---|---|---|---|---|---|
| **1.** | +100' Roof | A - B / 4 - 7 | 24" | 100 | Diaphragm |
| **2.** | + 80' Roof | (a) B - D / 1 - 7<br>(b) A - D / 1 - 7<br>(c) A - B / 2 – 3 | 24" | 80 | Diaphragm |
| **3.** | + 40' Floor | B -C / 1 - 2 | 18" | 40a | Diaphragm |
| **4.** | + 40' Floor | C - D / 4 - 6<br>A - B / 1 – 4 | 24" | 40b | Diaphragm |
| **5**. | + 32' Floor | A - B / 4 - 7<br>A – B / 6 -7 | 48" | 32 | Diaphragm |
| -- | + 20' Floor | B - C / 1 - 2 | 18" | 40a | Slab design |

# 2. REFERENCES

## 2.1 PROCEDURES/DIRECTIVES

2.1.1 BSC (Bechtel SAIC Company) 2007. EG-PRO-3DP-G04B-00037, Rev.9, *Calculations and Analyses.* Las Vegas, Nevada: ACC: ENG.20070420.0002

2.1.2 BSC (Bechtel SAIC Company) 2007. IT-PRO-0011, Rev. 07, ICN 0. *Software Management.* Las Vegas, Nevada: ACC: DOC.20070905.0007

2.1.3 ORD (Office of Repository Development) 2007, *Repository Project Management Automation Plan.* 000-PLN-MGR0-00200-000, Rev. 00E. Las Vegas, Nevada: U.S. Department of Energy, Office of the Repository Development. ACC: ENG.20070326.0019.

## 2.2 DESIGN INPUTS

2.2.1 BSC (Bechtel SAIC Company) 2006. *Project Design Criteria Document*. 000-3DR-MGR0-00100-000-006. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061201.0005

2.2.2 MacGregor, J.G. 1997. *Reinforced Concrete, Mechanics and Design*. Prentice Hall International Series in Civil Engineering and Engineering Mechanics. 3rd Edition. Upper Saddle River, N.J: Prentice Hall. TIC: 242587. [DIRS 130532]

2.2.3 ACI 349-01. 2001. *Code Requirements for Nuclear Safety Related Concrete Structures* (ACI 349-01). Farmington Hills, Michigan: American Concrete Institute. TIC: 252732. [DIRS 158833]

2.2.4 Not used.

2.2.5 BSC (Bechtel SAIC Company) 2006, *Basis of Design for the TAD Canister-Based Repository Design Concept.* 000-3DR-MGR0-00300-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061023.0002.

2.2.6 BSC (Bechtel SAIC Company) 2006. *Seismic Analysis and Design Approach Document.* 000-30R-MGR0-02000-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20061214.0008

2.2.7 Not used.

2.2.8 Not used.

2.2.9 Not Used .

2.2.10 Not Used .

2.2.11 Not Used.

2.2.12 Not Used.

2.2.13 BSC (Bechtel SAIC Company) 2007. *Tier 1 Seismic Analysis Using a Multiple Stick Model of the WHF*. 050-SYC-WH00-00200-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070326.0034

2.2.14 Not Used.

2.2.15 Not Used.

2.2.16 Not Used.

2.2.17 Not Used.

2.2.18 BSC (Bechtel SAIC Company) 2007. *Wet Handling Facility (WHF) Mass Properties* 050-SYC-WH00-00300-000-00B. Las Vegas, NV: Bechtel SAIC Company. ACC: ENG.20070326.0001.

## 2.3 DESIGN CONSTRAINTS

None

## 2.4 DESIGN OUTPUTS

Results of this calculation will be used in preparing preliminary WHF structural concrete slab, roof and diaphragm drawings. The drawing numbers have not yet been assigned to these drawings.

# 3. ASSUMPTIONS

## 3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 **Structural Steel Framing Dead Load (SFDL):** 40 lbs/ft$^2$ (at El. +100', +80', +40').

**Rationale:** This is a reasonable assumption for this preliminary slab and diaphragm calculation. The actual steel weights will be used as the design matures in the detailed design phase of the project. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2.

3.1.2 **Equipment Dead Load (EDL):** 100 lbs/ft$^2$ (at El. +40' and +32').
10 lbs/ft$^2$ (at Roof El. +100', +80').

Equipment dead loads include HVAC equipment, electrical equipment, and mechanical handling; equipment, hanging equipment, ducts, conduits, cable trays, etc.

**Rationale:** The WHF is not an equipment intensive structure with the major equipment for diaphragm design being the HVAC equipment. 100 lbs/ft$^2$ and 10 lbs/ft$^2$ are a reasonable assumption for this type of structure. Actual equipment weights will be used as the design matures in the detailed design phase of the project. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2

3.1.3 **Roofing Material Dead Load(RDL):** 55 lbs/ft$^2$ (at El. +100',+80')

This load allows for a light weight concrete fill material to be applied over the concrete slab with an average 6" thickness as well as membrane roofing material.

**Rationale:** This is a reasonable assumption that allows for a light weight concrete fill material to be applied over the concrete slab with an average thickness of 6 inches including a waterproof roofing membrane. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2

3.1.4 Not used

3.1.5 **Floor Live Load (FLL):** 100 lbs/ft$^2$ (25 lbs/ft$^2$ considered during EQ)

**Roof Live Load (RLL):** 40 lbs/ft$^2$ (10 lbs/ft$^2$ considered during EQ)

**Rationale:** 100 lbs/ft$^2$ live load for floor and 40 lbs/ft$^2$ live load for roof is a standard engineering practice for heavy industrial buildings and types of functions being performed in the WHF. Consideration of 25% of live load during seismic event is consistent with current revision of 000-30R-MGR0-02000-000, December 2006, *Seismic Analysis and Design Approach Document* (Ref. 2.2.6, Section 8.3.1). These loads are based on standard engineering practice for the type of structure. These loads are for use in a preliminary analysis only and will be refined in the detail design phase when specific equipment and operations being performed on each floor are better defined. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2.

3.1.6 **Floor slabs construction:** The concrete floor slabs (except 48" slab) are constructed on a 3" metal deck which are assumed to have maximum span of 7 feet.

**Rationale:** 7' is considered to be a reasonable span for a concrete floor slab. In the detailed design the actual maximum slab span will be used for the slab design. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2.

3.1.7 The amplified slab acceleration for out-of-plane seismic loads is assumed as 2.0 times the slab acceleration in vertical direction obtained from the Tier 1 WHF seismic analysis (Ref. 2.2.13).

**Rationale:** The Tier 1 seismic analysis model did not include the effects of vertical floor flexibility, i.e. the floors were considered as rigid diaphragms. Two times the vertical acceleration of slab is assumed to be appropriate. This assumption will be validated in the Tier 2 SSI analysis. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2.9

3.1.8 The WHF plans from the Plant Design Model as shown in Ref. 2.2.13 form the basis for defining the building layout plans and sketches as used in Ref. 2.2.13 are shown in Attachment A

**Rationale:** The development of the general arrangements continue to be refined; however the major rooms, wall locations and wall openings are adequately defined for design. The rationale for this assumption is that further refinement of the general arrangement will not significantly affect the structure. This calculation is adequate for slabs and diaphragms preliminary design. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used in Section 6.2, 6.3 and 6.4.

3.1.9 The floor slab for the 200-ton crane maintenance area between column lines B/C and 1/2, shown at El. +50' on Sketch Page A3 is relocated to El. +40'. Likewise, the floor slab supporting the pool equipment is relocated from El. +30' as shown in the sketch to El. +20'. Relocation of the two slabs will be incorporated in the plant design drawings.

**Rationale:** Relocating the crane maintenance slab to El. +40' is to provide continuity to the frame diaphragm resulting in a more stable building structure. Crane maintenance function will not be impacted by this relocation. The pool equipment floor is then conveniently relocated in the middle of the crane maintenance floor and the ground floor at El. +0'. This assumption is being tracked in CalcTrac.

**Where used**: Assumption used on Page A3 of Attachment A

3.1.10 North South Diaphragm at Elevation 80 Ft.

The concrete roof slab at elevation 80 ft. is a diaphragm assumed to be from Column Line 1 to 7 (266 ft.) with a depth between Column Lines B and D (157 ft.). See Attachment B, page B2. The diaphragm is acting as a deep beam with continuous spans between the exterior and interior walls.

**Rationale:** Since internal shear walls provide support for the diaphragm, the largest spacing between walls will be conservatively taken as the beam span to calculate the in-plane bending moment. This moment will be used to determine the preliminary diaphragm chord reinforcement from Column Line 1 to 7. This diaphragm and supporting shear walls will be included in a detailed finite element model for the static and dynamic analysis of the WHF that will supersede this calculation. This calculation is being tracked in CalcTrac.

**Where used:** Assumption used in Section 6.2.

3.1.11 In plane slab/diaphragm loads due to the 200-ton crane and Canister Transfer Machine (CTM) are relatively insignificant compared to the total in-plane loads from the weights of the concrete slab, distributed dead and live loads and concrete walls. The 200-ton crane and CTM will have a negligible impact on the diaphragm analysis and are not include in this preliminary design.

**Rationale**: Because of the relatively small weight increase, this is a reasonable assumption for this preliminary design calculation. This assumption will be validated in the Tier 2 finite element analysis that will include both pieces of equipment in the total building mass. This assumption will be traced in CalcTrac.

**Where used:** Assumption used in Section 6.4.4.

## 3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 48" thick floor slab bounded by Col. lines A – B / 4 – 7 at El. +32' is assumed to have fixed supports on two edges and is designed as one-way slab.

**Rationale**: Using fixed supports is a reasonable assumption and designing one-way slab instead of two-way slab is bounding. Reinforcing steel computed in the slab span direction will also be provided in the orthogonal direction.

**Where used**: Assumption used in Section 6.2.

3.2.2 Any multiple span diaphragms when analyzed for in-plane loads are taken as simple spans using the largest span.

**Rationale:** Taking simple span instead of multiple spans is conservative because moment will be larger in magnitude and is acceptable.

**Where used**: Assumption used in Section 6.2.

3.2.3 The lever arm distance for chord steel in diaphragm (see Attachment C) is considered as 0.90 times the length /or the width of the diaphragm for calculating the chord force (see Section 6.6.1 and 6.6.2)

**Rationale:** The chord steel is located within the slab at the center of the 4' thick wall below. Center-to-center distance between the centers of 4' thick walls is always greater than 0.9d. Therefore, 0.9d is a conservative and a bounding assumption for the WHF.

**Where used**: Assumption used in Section 6.2.

3.2.4. The total reinforcing steel for the slabs/diaphragms is obtained by summing the required steel for out of plane loads with the required steel for in-plane loads. This is a conservative method for determining the total reinforcing steel for slabs/diaphragms.

**Rationale:** This is a conservative, bounding assumption for this preliminary design. Detailed design using the results of the Tier 2 finite element static and dynamic analyses will comply with the requirements of the *Seismic Analysis and Design Document,* Appendix A, Ref. 2.2.6. This methodology response combination, including earthquake induced loads from different directions, uses the 100 % of maximum load from one direction with 40% of the maximum loads from the other two component directions.

**Where used**: Assumption used in Section 6.5.

# 4. METHODOLOGY

## 4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with EG-PRO-3DP-G04B-00037, Rev. 9, *Calculations and Analyses* (Ref. 2.1.1). Section 5.1.2 of the *Basis of Design for the TAD*

*Canister-Based Repository Design Concept* (Ref. 2.2.5) classifies the WHF structure as ITS. The approved record version of this document is designated QA:QA.

## 4.2 USE OF SOFTWARE

Word 2003, part of the Microsoft Office 2003 suite of programs, was used in this calculation. Microsoft Office 2003 is classified as Level 2 software as defined in IT-PRO-0011, *Software Management*, (Ref 2.1.2). Microsoft Office 2003 is listed on the current Software Report. Microsoft Office software is also listed in 000-PLN-MGR0-00200-000, *Repository Project Management Automation Plan*, (Ref. 2.1.3).

Mathcad 13 was utilized to perform mathematical computations in this calculation. Mathcad 13 is classified as Level 2 software as defined in IT-PRO-0011, Software Management,, (Ref 2.1.2). Mathcad 13 is listed on the current Software Report. The Mathcad software is also listed in 000-PLN-MGR0-00200-000, *Repository Project Management Automation Plan*, (Ref. 2.1.3). The verification of the Mathcad 13 computations in this calculation is done using a hand calculator.

Software was executed on a PC system running Microsoft Windows 2000 operating system.

The calculation process and equations are documented in Section 6 of this calculation for checking by manual calculations.

## 4.3 DESIGN APPROACH

In this calculation, a representative sample of slabs (Cases 1 thru 5) will be designed that cover all the diaphragms at different elevations (see Attachment B):

These sample slab panels include the following Cases for design purposes:

| | | Panel Locations | Design Panels |
|---|---|---|---|
| **Case 1**: | 24" Thick-Roof Diaphragm at El. 100': | A - B / 4 - 7 | A - B / 4 - 7 |
| **Case 2**: | 24" Thick-Roof Diaphragm at El. 80': | A - D / 1 – 7 | (a) B - D / 1 - 7<br>(b) A - D / 1 - 7<br>(c) A - B / 2 - 3 |
| **Case 3**: | 18" Thick-Floor Diaphragm at El. 40': | B - C / 1 – 2 | B – C / 1 – 2 |
| **Case 4**: | 24" Thick-Floor Diaphragm at El. 40': | C - D / 1 - 6<br>A - B / 1 - 4 | C - D / 4 - 6<br>A - B / 1 - 4 |
| **Case 5**: | 48" Thick-Floor Diaphragm at El. 32': | A - B / 4 - 7 | A - B / 4 - 7<br>A - B / 6 - 7 |

For the Wet Handling Facility, the diaphragm elevations are located at El. +100', +80', +40' and +32'. Masses of the walls are lumped at the diaphragms by considering that half of the wall mass is tributary to the floor at the bottom of the wall and half of the mass is tributary to the floor at the top of the wall. This methodology is consistent with the methodology commonly used in the development of the diaphragm design.

In this calculation, five cases are selected to design all the diaphragms to cover all the building floors at different elevations (see Attachment B).

Case 1: The slab diaphragm 24" thick at elev. +100' is supported by exterior walls only and is designed for N/S and E/W acceleration loadings.

Case 2: This is a 24" thick diaphragm at elev. +80' covering larger area and supported by: exterior and interior walls. This slab is designed in three portions for governing load conditions:

( i ) For N/S acceleration loading, the panel B – D / 1 - 7 is considered 157' deep with a longest span of 64',

(ii) For N/S acceleration loading, the panel A – B / 2 - 3 is considered 53' deep with a span of 54' (see Section 6.6.), and

(iii) For E/W loading, the panel A–D / 1-7 is considered 266' deep and the longest span is considered as 104' ( B to C ) to get the maximum moments and the chord reinforcement in the diaphragm panels.

Case 3: This is a 18" thick diaphragm (B–C / 1–2) at elev. + 40' supported by exterior walls only, and is designed for N/S and E/W acceleration loadings.

Case 4: This is a 24" thick diaphragm at elev. +40' covering larger area and supported by exterior and interior walls above and below the diaphragm. This slab is also designed in two portions for different load conditions- (i) For N/S acceleration loading, the panel C –D / 4 -6 is considered with a span of 64' and depth of 53' and (ii) for E/W acceleration loading the panel A – B / 1 - 4 has a span of 53' and depth of 148'. The same reinforcement is provided in the panel C –D / 1 -6.

Case 5: This is a 48" thick diaphragm at elev. +32' covering area A – B / 4 -7. This panel is supported by exterior walls and interior walls. This panel is also designed in two portions for different load conditions. (i) For N/S acceleration loading, the panel A – B / 6 -7 is considered with a span of 54' and a depth of 53', and (ii) for E/W acceleration loading the panel A – B/4 – 7 is considered with a span of 53' and a depth of 118'.

1. Exterior wall used here is for the walls located at the edge of the slab/diaphragm panel.

2. Interior wall used here for the walls located at the interior of the slab/diaphragm panel.

The slab configurations are based on preliminary concrete outline sketches (Attachment A)

The concrete slabs/diaphragms will be designed for the vertical floor loads (i.e. dead loads, live loads, equipment loads, etc.) applied to the slab simultaneously with the in-plane and out-of-plane loads imposed on the slabs under seismic loading conditions.

The weight of the tributary interior and exterior walls will be applied to diaphragms consistent with the way tributary wall load masses were located at the diaphragm levels in the Tier 1 lumped mass, multiple stick model. For E/W acceleration loading, the weights of all interior walls normal to E/W acceleration are summed at each diaphragm level, similarly, for N/S acceleration loading, the weights of all interior walls normal to N/S acceleration are summed at each diaphragm level. A uniform in-plane load is computed using the length of the building normal to the earthquake direction.

Section 5.1.2 of the *Basis of Design for the TAD Canister–Based Repository Design Concept* (Ref.2.2.5) classifies the WHF structure as ITS, therefore the design will be based on the requirements of ACI 349 *Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-01) and Commentary (ACI 349R-01)*, hereinafter referred to as ACI 349 (Ref. 2.2.3)

Diaphragms transmit the horizontal seismic loads to the shear walls. In-plane diaphragm loads are a result of horizontal seismic acceleration of the diaphragm itself and the walls tributary to the diaphragm perpendicular to the direction of the seismic acceleration. For example, under a North to South seismic acceleration a diaphragm must transfer horizontal seismic load equal to the mass of the diaphragm plus the mass of the East to West walls (tributary to the diaphragm) times the horizontal seismic acceleration to the North to South shear walls.

The diaphragm design is carried out in three steps.

- Reinforcing requirements for out-of-plane (bending) loads are calculated.
- Reinforcing requirements for in-plane (diaphragm) shear loads are calculated.
- Reinforcing requirements for in-plane (diaphragm) moments are calculated.

The results of the first and second steps are combined to determine the reinforcing requirements for the out-of-plane bending and in-plane shear loads. These reinforcing requirement are then compared to the ACI 349 (Ref. 2.2.3) minimum reinforcing requirements. The larger of the reinforcing required for the out-of-plane bending and in-plane shear loads and minimum requirements will determine the reinforcing requirement.

The results of the third step (in-plane moments) yield the chord steel required for the diaphragm.

# 5. LIST OF ATTACHMENTS

## 6.0 BODY OF CALCULATIONS

## 6.1 UNITS, STRESSES AND VARIABLES

### 6.1.1 Units :

$Plf := \frac{lbf}{ft}$ pounds per square foot $Psf := \frac{lbf}{ft^2}$ pounds per cubic foot $Pcf := \frac{lbf}{ft^3}$

$kft := \frac{kip}{ft}$ kip : 1000 lbs klf : kip per linear foot psi : pounds per square inch

### 6.1.2 Stresses

$f_c := 5000 \cdot psi$ Compressive Strength of Concrete (Ref. 2.2.1, Section 4.2.11.6.2)

$f_y := 60000 \cdot psi$ Yield Stress of Grade 60 Reinforcing Steel (Ref. 2.2.1, Section 4.2.11.6.2)

### 6.1.3 Variables

| | |
|---|---|
| As | Area of reinforcing steel (in$^2$) |
| b | Width of concrete section (inches) |
| clr | Clear cover over reinforcing bars (inches) |
| d | Effective depth of reinforcing (inches) |
| $d_{bar}$ | Diameter of reinforcing bar (inches) |
| h | Height of section or slab thickness (inches) |
| span | Span of beam (feet) |
| w | Uniform applied load on beam (lb/ft) |
| $\phi_s$ | Strength reduction factor for shear |
| $\phi_b$ | Strength reduction factor for bending |
| $\phi_{diaph}$ | Strength reduction factor for in-plane shear (diaphragm shear) |
| $\rho$ | Reinforcing ratio = As/(b*d) |
| $\rho_{req}$ | Computed required reinforcing ratio |
| $\omega$ | Reinforcing index = $\rho * (f_y / f_c)$ |

| | |
|---|---|
| $A_{ch}$ | Required Chord Reinforcment Area ($in^2$) |
| $A_{cv}$ | Shear Area ($ft^2$) |
| C | Constant = $\omega$ (1 - 0.59$\omega$) |
| CF | Chord Force (kips) |
| E | Seismic Load |
| EDL | Equipment Dead Load |
| LL | Live Load |
| lw | Length of Wall |
| M | Moment |
| RMDL | Roofing Material Dead Load |
| SACC | Seismic Acceleration |
| SDL | Slab Dead Load |
| SFDL | Steel Framing Dead Load |
| TDL | Total Dead Load |
| Tw | Thickness of Wall (ft) |
| U | Ultimate Load |
| Vn | Nominal Shear Strength = (Vc + Vs) |
| $V_c$ | Concrete Shear Strength |
| Vs | Reinforcing Steel Shear Strength |
| W | Load per Feet |
| WW | Weight of Wall |

## 6.2 DESIGN LOADS

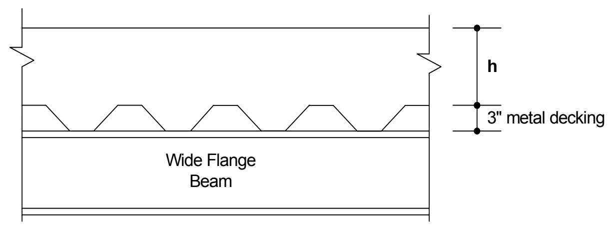
### 6.2.1 Concrete Slabs Dead Load (SDL):

$w_{conc} := 150 \cdot Pcf$ Unit weight of concrete (Reference 2.2.1, Section 4.2.11.6.6)

The dead weight of a concrete slab constructed on a 3"( 0.25 ft ) metal deck ( 18" & 24" thick slabs): (see Assumption 3.1.6)

$$SDL := \left[\left(\frac{h \cdot in}{12} + \frac{0.25 \cdot ft}{2}\right) \cdot w_{conc}\right]$$

where h is the slab thickness above the metal decking in inches




**Locations of slabs/Diaphragms to be designed in this calc:** (See Attachment B)

| <u>Cases</u> | Elevation | Between Column lines | Thick | Mathcad Subscript | Design as |
|---|---|---|---|---|---|
| **1.** | +100' Roof | A - B / 4 - 7 | 24" | 100 | Diaphragm |
| **2.** | + 80' Roof | (a) B - D / 1 - 7<br>(b) A - D / 1 - 7<br>(c) A - B / 2 – 3<br>( Note: For the design of c above, see Section 6.6) | 24" | 80 | Diaphragm |
| **3.** | + 40' Floor | B -C / 1 - 2 | 18" | 40a | Diaphragm |
| **4.** | + 40' Floor | C - D / 4 - 6<br>A - B / 1 – 4 | 24" | 40b | Diaphragm |
| **5.** | + 32' Floor | A - B / 6 - 7<br>A –B / 4 – 7 | 48" | 32 | Diaphragm |
| | + 20' Floor | B - C / 1 - 2 | 18" | 40a | Slab design (Not a Diaphragm) |

24" thick slab at +100' Roof $SDL_{100} := (24 \cdot in + 1.5 \cdot in) \cdot w_{conc}$ $SDL_{100} = 319\,Psf$

24" thick slab at +80' Roof $SDL_{80} := (24 \cdot in + 1.5 \cdot in) \cdot w_{conc}$ $SDL_{80} = 319\,Psf$

18" thick slab at +40' $SDL_{40a} := (18 \cdot in + 1.5 \cdot in) \cdot w_{conc}$ $SDL_{40a} = 244\,Psf$

24" thick slab at +40' $SDL_{40b} := (24 \cdot in + 1.5in) \cdot w_{conc}$ $SDL_{40b} = 319\,Psf$

48" thick slab at +32' $SDL_{32} := (48 \cdot in) \cdot w_{conc}$ $SDL_{32} = 600\ Psf$

Note : 48" thick slab does not have metal deck or beam under the slab, but supported by conc. walls underneath.

$$SDL := \begin{pmatrix} SDL_{100} \\ SDL_{80} \\ SDL_{40a} \\ SDL_{40b} \\ SDL_{32} \end{pmatrix}$$

Slab Dead Load : 24" slab
Slab Dead Load : 24" slab
Slab Dead Load : 18" slab
Slab Dead Load : 24" slab
Slab Dead Load : 48" slab

$$\boxed{SDL = \begin{pmatrix} 319 \\ 319 \\ 244 \\ 319 \\ 600 \end{pmatrix} Psf}$$

### 6.2.2 Equipment Dead Load (EDL): ( see Assumption 3.1.2 )

EDL = 10 Psf at +100', +80'

EDL =100 Psf at +40', +32'

$EDL_{100} := 10Psf$

$EDL_{80} := 10Psf$

$EDL_{40a} := 100Psf$

$EDL_{40b} := 100Psf$

$EDL_{32} := 100Psf$

$$EDL := \begin{pmatrix} EDL_{100} \\ EDL_{80} \\ EDL_{40a} \\ EDL_{40b} \\ EDL_{32} \end{pmatrix}$$

Equipment Dead Load at 24" roof slab
Equipment Dead Load at 24" roof slab
Equipment Dead Load at 18" floor slab
Equipment Dead Load at 24" floor slab
Equipment Dead Load at 48" floor slab

$$\boxed{EDL = \begin{pmatrix} 10 \\ 10 \\ 100 \\ 100 \\ 100 \end{pmatrix} Psf}$$

### 6.2.3 Not used

### 6.2.4 Struct. Steel Framing Dead Load (SFDL): ( Assumption 3.1.1 )

SFDL = 40 Psf at +100', +80', +40'

SFDL = 0 Psf at + 32'

$SFDL_{100} := 40Psf$

$SFDL_{80} := 40Psf$

$SFDL_{40a} := 40Psf$

$SFDL_{40b} := 40Psf$

$SFDL_{32} := 0Psf$

$$SFDL := \begin{pmatrix} SFDL_{100} \\ SFDL_{80} \\ SFDL_{40a} \\ SFDL_{40b} \\ SFDL_{32} \end{pmatrix}$$

Steel Framing Dead Load : 24" roof slab
Steel Framing Dead Load : 24" roof slab
Steel Framing Dead Load : 18" floor slab
Steel Framing Dead Load : 24" floor slab
Steel Framing Dead Load : 48" floor slab

$$\boxed{SFDL = \begin{pmatrix} 40 \\ 40 \\ 40 \\ 40 \\ 0 \end{pmatrix} Psf}$$

**6.2.5 Roofing Material Dead Load (RMDL)**(6"- lightweight concrete fill, see Assumption 3.1.3)

RMDL = 55 Psf at +100', +80'

$RMDL_{100} := 55Psf$
$RMDL_{80} := 55Psf$
$RMDL_{40a} := 0Psf$
$RMDL_{40b} := 0Psf$
$RMDL_{32} := 0psf$

$$RMDL := \begin{pmatrix} RMDL_{100} \\ RMDL_{80} \\ RMDL_{40a} \\ RMDL_{40b} \\ RMDL_{32} \end{pmatrix}$$

Roofing Material Dead Load : 24" roof slab
Roofing Material Dead Load : 24" roof slab
Roofing Material Dead Load : 18" floor slab
Roofing Material Dead Load : 24" floor slab
Roofing Material Dead Load : 48" floor slab

$$\boxed{RMDL = \begin{pmatrix} 55 \\ 55 \\ 0 \\ 0 \\ 0 \end{pmatrix} Psf}$$

**6.2.6 Roof Live Load (RLL):** ( see Assumption 3.1.5 ) (Use 25% of live load in combination with seismic loads)

RLL = 40 Psf at +100', +80'

$RLL_{100} := 40Psf$
$RLL_{80} := 40Psf$
$RLL_{40a} := 0Psf$
$RLL_{40b} := 0Psf$
$RLL_{32} := 0Psf$

$$RLL := \begin{pmatrix} RLL_{100} \\ RLL_{80} \\ RLL_{40a} \\ RLL_{40b} \\ RLL_{32} \end{pmatrix}$$

Roof Live Load : 24" roof slab
Roof Live Load : 24" roof slab
Roof Live Load : 18" Floor slab
Roof Live Load : 24" Floor slab
Roof Live Load : 48" Floor slab

$$\boxed{RLL = \begin{pmatrix} 40 \\ 40 \\ 0 \\ 0 \\ 0 \end{pmatrix} Psf}$$

**6.2.7 Floor Live Load (FLL):** ( see Assumption 3.1.5 ) (Use 25% of live load in combination with seismic loads)

FLL = 100 Psf at +40', +32'

$FLL_{100} := 0Psf$

$FLL_{80} := 0Psf$

$FLL_{40a} := 100Psf$

$FLL_{40b} := 100Psf$

$FLL_{32} := 100Psf$

$$FLL := \begin{pmatrix} FLL_{100} \\ FLL_{80} \\ FLL_{40a} \\ FLL_{40b} \\ FLL_{32} \end{pmatrix}$$

Floor Live Load : 24" roof slab
Floor Live Load : 24" roof slab
Floor Live Load : 18" Floor slab
Floor Live Load : 24" Floor slab
Floor Live Load : 48" Floor slab

$$\boxed{FLL = \begin{pmatrix} 0 \\ 0 \\ 100 \\ 100 \\ 100 \end{pmatrix} Psf}$$

**6.2.8 Total Dead Load (TDL):**

$TDL_{100} := SDL_{100} + EDL_{100} + SFDL_{100} + RMDL_{100}$

$TDL_{80} := SDL_{80} + EDL_{80} + SFDL_{80} + RMDL_{80}$

$TDL_{40a} := SDL_{40a} + EDL_{40a} + SFDL_{40a} + RMDL_{40a}$

$TDL_{40b} := SDL_{40b} + EDL_{40b} + SFDL_{40b} + RMDL_{40b}$

$TDL_{32} := SDL_{32} + EDL_{32} + SFDL_{32} + RMDL_{32}$

$$TDL := \begin{pmatrix} TDL_{100} \\ TDL_{80} \\ TDL_{40a} \\ TDL_{40b} \\ TDL_{32} \end{pmatrix}$$

$$\boxed{TDL = \begin{pmatrix} 424 \\ 424 \\ 384 \\ 459 \\ 700 \end{pmatrix} Psf}$$

### 6.2.9 Acceleration Factors for Seismic Loads at elev.+100', +80', +40', +32'
( Ref: 2.2.13, Table 18, g- values ):

Note:The amplified slab acceleration for out-of- plane seismic loads is 2.0 ( see Assumption 3.1.7 )

Accelerations for elevation+ 100' :

$a100_x := 1.34$ (East/West)

$a100_y := 1.627$ (North/South)

$a100_z := 2 \cdot 0.88$ (Vertical)

Accelerations for elevation + 80':

$a80_x := 0.987$ (East/West)

$a80_y := 0.978$ (North/South)

$a80_z := 2 \cdot 0.707$ (Vertical)

Accelerations for elevation + 40':
(For both Cases 40a and 40b)

$a40_x := 0.733$ (East/West)

$a40_y := 0.744$ (North/South)

$a40_z := 2 \cdot 0.660$ (Vertical)

Accelerations for elevation + 32':

$a32_x := 0.741$ (East/West)

$a32_y := 0.692$ (North/South)

$a32_z := 2 \cdot 0.722$ (Vertical)

**(i) Seismic Load ( E ) in Horizontal X and Y -Directions: ( For co-ordinate system, see Attachment A )**

$$SACC_x := \begin{pmatrix} a100_x \\ a80_x \\ a40_x \\ a40_x \\ a32_x \end{pmatrix} \quad \begin{array}{l} \text{Seismic Acceleration for 24" slab @ El 100' in X direction} \\ \text{Seismic Acceleration for 24" slab @ El 80' in X direction} \\ \text{Seismic Acceleration for 18" slab @ El 40' in X direction} \\ \text{Seismic Acceleration for 24" slab @ El 40' in X dir.} \\ \text{Seismic Acceleration for 48" slab @ El 32' in X direction} \end{array} \quad SACC_x = \begin{pmatrix} 1.34 \\ 0.987 \\ 0.733 \\ 0.733 \\ 0.741 \end{pmatrix}$$

$$SACC_y := \begin{pmatrix} a100_y \\ a80_y \\ a40_y \\ a40_y \\ a32_y \end{pmatrix} \quad \begin{array}{l} \text{Seismic Acceleration for 24" slab @ El 100' in Y direction} \\ \text{Seismic Acceleration for 24" slab @ El 80' in Y direction} \\ \text{Seismic Acceleration for 18" slab @ El 40' in Y direction} \\ \text{Seismic Acceleration for 24" slab @ El 40' in Y dir.} \\ \text{Seismic Acceleration for 48" slab @ El 32' in Y direction} \end{array} \quad SACC_y = \begin{pmatrix} 1.627 \\ 0.978 \\ 0.744 \\ 0.744 \\ 0.692 \end{pmatrix}$$

**(ii) Seismic Load ( E ) in Vertical Z- Direction.**

$$SACC_z := \begin{pmatrix} a100_z \\ a80_z \\ a40_z \\ a40_z \\ a32_z \end{pmatrix}$$

Seismic Acceleration for 24" slab @ El 100' in Z direction
Seismic Acceleration for 24" slab @ El 80' in Z direction
Seismic Acceleration for 18" slab @ El 40' in Z direction
Seismic Acceleration for 24" slab @ El 40' in Z dir.
Seismic Acceleration for 48" slab @ El 32' in Z direction

$$\boxed{SACC_z = \begin{pmatrix} 1.760 \\ 1.414 \\ 1.320 \\ 1.320 \\ 1.444 \end{pmatrix}}$$

**6.2.10 Load Combinations :**

$$LL := RLL + FLL$$

$$LL = \begin{pmatrix} 40 \\ 40 \\ 100 \\ 100 \\ 100 \end{pmatrix} \text{Psf}$$

$U1 := 1.4 \cdot TDL + 1.7 \cdot LL$ (Ultimate Load for normal operating condition) (Ref. 2.2.1, Section 4.2.11.4.5)

$U2 = TDL + LL + E$ (Ultimate Load, use 25% of live load in combination with seismic loads) (Ref. 2.2.6, Section 8.3.1 )

## 6.3 SLAB DESIGN FOR OUT-OF-PLANE (VERTICAL) LOADS:

Refer to the plant design drawings (Ref. 2.2.9 to 2.2.12) - for plans and sections of the WHF-structure and Attachment B of this calculation.

All 24" and 18" thick floor/roof slabs are constructed on a 3"- metal deck.

**Slab A** : Floor slabs constructed on a 3" metal deck with a maximum span of 7'-0" (see Assumption 3.1.6 ) ( 24" thick slab at elev. +100', +80', +40' and 18" thick slab at elev. +40' & +20')

**Slab B** : Fixed support on two edges for 48"-slab at elev. +32' ( see Assumption 3.2.1) bounded by Col. lines A - B / 4 - 7 (Ref. 2.2.10 and Attachment B of this calculation).

For these cases, the effective depth of the slab, d, can be calculated as:

where h is the slab thickness above the metal deck

$$d = h - 0.75\text{" (clr)} - 1.50x\ d_{bar}$$

For a # 11 reinforcing bar, $d_{bar}$ = 1.41", this results in an effective rebar depth,d, of 15.13" for 18" slab, 21.13" for 24" slab and 45.13" for 48" slab.
Note: # 11reinforcing bar is used to determine the least effective depth for concrete slab design.

$d_{18} := 15.13\text{in}$ $d_{24} := 21.13\text{in}$ $d_{48} := 45.13\text{in}$ $\text{span} := 7\text{ft}$

(see Assumption 3.1.6)

### 6.3.1 Total Dead Load (TDL): ( From Sect. 6.2.8 )

$$TDL := \begin{pmatrix} TDL_{100} \\ TDL_{80} \\ TDL_{40a} \\ TDL_{40b} \\ TDL_{32} \end{pmatrix} \qquad TDL = \begin{pmatrix} 424 \\ 424 \\ 384 \\ 459 \\ 700 \end{pmatrix} Psf$$

### 6.3.2 Seismic Load ( E ) ( Vertical in Z-dir ) :

$$\boxed{E_z := \overrightarrow{\left[(TDL + 0.25 \cdot LL) SACC_z\right]}} \qquad E_z = \begin{pmatrix} 763 \\ 613 \\ 540 \\ 639 \\ 1047 \end{pmatrix} Psf$$

Note: Use 25% of LL in combination with seismic loads
( see Ref. 2.2.6, Section 8.3.1)

$ORIGIN \equiv 1$ Redefines origin of matrix

$E_{100} := E_{z_1}$ Seismic Load for 24" slab @ El 100' in Z direction $E_{100} = 763\,Psf$

$E_{80} := E_{z_2}$ Seismic Load for 24" slab @ El 80' in Z direction $E_{80} = 613\,Psf$

$E_{40a} := E_{z_3}$ Seismic Load for 18" slab @ El 40' in Z direction $E_{40a} = 540\,psf$

$E_{40b} := E_{z_4}$ Seismic Load for 24" slab @ El 40' in Z direction $E_{40b} = 639\,Psf$

$E_{32} := E_{z_5}$ Seismic Load for 48" slab @ El 32' in Z direction $E_{32} = 1047\,Psf$

### 6.3.3 Governing Ultimate Load Combination for Concrete Design:

From Sect. 6.2.10 of this calculation.

$U1 := 1.4 \cdot TDL + 1.7 \cdot LL$

For 24" thick roof slab at El. +100'
For 24" thick roof slab at El. +80'
For 18" thick slab at El. +40'
For 24" thick slab at El. +40'
For 48" thick slab at El. +32'

$$U1 = \begin{pmatrix} 661 \\ 661 \\ 707 \\ 812 \\ 1150 \end{pmatrix} Psf$$

$\boxed{U2 := TDL + LL + E_z}$

For 24" thick roof slab at El. +100'
For 24" thick roof slab at El. +80'
For 18" thick slab at El. +40'
For 24" thick slab at El. +40'
For 48" thick slab at El. +32'

$$U2 = \begin{pmatrix} 1227 \\ 1077 \\ 1023 \\ 1197 \\ 1847 \end{pmatrix} Psf$$

**U2 > U1, Therefore, the governing Design Loads are U2**.

### 6.3.4 Determine MOMENTS and SHEARS for Slab A and Slab B:

#### 6.3.4.1 MOMENTS and SHEARS for Slab A:

Using the moment coefficients from ACI 349 ( Ref: 2.2.3, Section 8.3.3) :

(a) Maximum positive (+) moment = ($wL^2/14$ )

(b) Maximum negative ( - ) moment at supports = ($wL^2/10$) ( spans <10' ) **Governs**

(c) Maximum shear force = (1.15wL/2) ( For end span) (End span governs)

(d) Maximum shear force = (wl/2) ( For 48" thick slab)

For a 1' strip of slab : $b := 1 \cdot ft$ width $Span := 7 \cdot ft$ $Span = 7.00000\ ft$
(Assumption 3.1.6)

$$U_A := \begin{pmatrix} U2_1 \\ U2_2 \\ U2_3 \\ U2_4 \end{pmatrix} \qquad U_A = \begin{pmatrix} 1227 \\ 1077 \\ 1023 \\ 1197 \end{pmatrix} Psf$$

24" roof slab at +100'
24" floor slab at + 80'
18" floor slab at + 40'
24" roof slab at + 40'

$W_{strip} := 1 \cdot ft$

$$\boxed{M_{max} := \overrightarrow{\frac{U_A \cdot W_{strip} \cdot span^2}{10}}}$$

i.e. Negative Moment

Moment 24" roof slab at +100'
Moment 24" floor slab at +80'
Moment 18" roof slab at +40'
Moment 24" roof slab at +40'
For 48" slab, see in Section 6.3.4.2

$$M_{max} = \begin{pmatrix} 6013 \\ 5278 \\ 5014 \\ 5867 \end{pmatrix} lbf \cdot ft \quad \text{per foot}$$

$$\boxed{V_{max} := \overrightarrow{\frac{1.15 \cdot U_A \cdot W_{strip} \cdot span}{2}}}$$

Shear 24" roof slab
Shear 24" roof slab
Shear 18" floor slab
Shear 24" floor slab

$$V_{max} = \begin{pmatrix} 4939 \\ 4335 \\ 4119 \\ 4819 \end{pmatrix} lbf \quad \text{per foot}$$

For 48" slab, see in Section 6.3.4.2

**6.3.4.2 Moment and Shear for 48" thk. Slab B at El. +32' :**

Take fixed ends at opposite sides with a span of 53' (between column lines A/4-7 and B/4-7).

Maximum shear force = (wl/2)

$span_{32} := 53 \cdot ft$ Span of 48" thk. floor slab

$U_{32} := U2_{5,1}$ (Case 5) $U_{32} = 1847\, Psf$ Ultimate Load for 48" floor slab

$$\boxed{M_{32} := \frac{U_{32} \cdot b \cdot span_{32}^2}{10}}$$

$M_{32} = 518794\, lbf \cdot ft$ per ft Max. Moment for 48" floor slab

$$\boxed{V_{32} := \frac{U_{32} \cdot b \cdot span_{32}}{2}}$$

$V_{32} = 48943\, lbf$ per ft Max. Shear for 48" floor slab

**6.3.4.3 Moments and Shears for Slabs A and Slab B:**

$$M_{AB} := \begin{pmatrix} M_{max_1} \\ M_{max_2} \\ M_{max_3} \\ M_{max_4} \\ M_{32} \end{pmatrix}$$

Moment 24" roof slab at +100'

Moment 24" floor slab at +80'

Moment 18" roof slab at +40'

Moment 24" roof slab at +40'

Moment 48" floor slab at +32'

$$M_{AB} = \begin{pmatrix} 6013 \\ 5278 \\ 5014 \\ 5867 \\ 518794 \end{pmatrix} lbf \cdot ft$$ per foot

$$V_{AB} := \begin{pmatrix} V_{max_1} \\ V_{max_2} \\ V_{max_3} \\ V_{max_4} \\ V_{32} \end{pmatrix}$$

Max Shear 24" roof slab at +100'

Max Shear 24" floor slab at +80'

Max Shear 18" roof slab at +40'

Max Shear 24" roof slab at +40'

Max Shear 48" floor slab at +32'

$$V_{AB} = \begin{pmatrix} 4939 \\ 4335 \\ 4119 \\ 4819 \\ 48943 \end{pmatrix} lbf$$ per foot

### 6.3.5 Check Shear Reinforcement Requirements:

( From Section 6.3 )

$$d := \begin{pmatrix} d_{24} \\ d_{24} \\ d_{18} \\ d_{24} \\ d_{48} \end{pmatrix} \qquad \boxed{d = \begin{pmatrix} 21.13 \\ 21.13 \\ 15.13 \\ 21.13 \\ 45.13 \end{pmatrix} in}$$

Effective depth of the rebar 24" roof slab

Effective depth of the rebar 24" roof slab

Effective depth of the rebar 18" floor slab

Effective depth of the rebar 24" floor slab

Effective depth of the rebar 48" roof slab

$$\boxed{h := \begin{pmatrix} 24 \\ 24 \\ 18 \\ 24 \\ 48 \end{pmatrix} \cdot in} \quad \& \quad b := 12 \cdot in$$

<u>Design Shear strength of concrete, $\phi s.Vc$:</u>

$\phi_s := .85$ Strength reduction factor for transverse shear (Ref: 2.2.3, Section 9.3.2.3)

$$\boxed{V_c := 2 \cdot \phi_s \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d}$$

(Ref: 2.2.3, Sect. 11.3.1.1)

Shear Strength of 24" roof slab at +100'

Shear Strength of 24" roof slab at +80'

Shear Strength of 18" floor slab at +40'

Shear Strength of 24" floor slab at +40'

Shear Strength of 48" floor slab at +32'

$$\boxed{V_c = \begin{pmatrix} 30480 \\ 30480 \\ 21825 \\ 30480 \\ 65100 \end{pmatrix} lbf}$$ per foot

**Since, $\phi$s.Vc > $V_{AB}$** (Section 6.3.4.3),

**No shear reinforcing required for 18", 24" and 48" slabs and the slabs are adequate for shear.**

### 6.3.6 Compute Bending Reinforcement Requirements:

The moment capacity of a reinforced concrete slab is computed as:

$$\boxed{Mu := \phi_b \cdot f_c \cdot b \cdot d^2 \cdot \omega \cdot (1 - .59 \cdot \omega)}$$ (Ref: 2.2.2 , Page 102, Eqn. 4-15)

Where $\omega := \dfrac{\rho \cdot f_y}{f_c}$ and $\rho := \dfrac{A_s}{b \cdot d}$

$\phi_b := .9$ Strength reduction factor for bending (Ref: 2.2.3, Section 9.3.2.1)

Re-arranging to solving for $\omega$: $\omega(1 - .59\omega) := \dfrac{M_u}{\phi_b \cdot f_c \cdot b \cdot d^2}$

Letting the right hand side of the equation to be a constant

$$\boxed{C_1 := \overrightarrow{\frac{M_{AB}}{\phi_b \cdot f_c \cdot b \cdot d^2}}}$$

Constants $C_1 = \begin{pmatrix} 0.002993 \\ 0.002627 \\ 0.004868 \\ 0.00292 \\ 0.056605 \end{pmatrix}$

| | | | |
|---|---|---|---|
| $C1 := C1_1$ | $C1 = 0.002993$ | Constant for 24" roof slab at +100' | Case 1 |
| $C2 := C1_2$ | $C2 = 0.002627$ | Constant for 24" floor slab at +80' | Case 2 |
| $C3 := C1_3$ | $C3 = 0.004868$ | Constant for 18" floor slab at +40' | Case 3 |
| $C4 := C1_4$ | $C4 = 0.002920$ | Constant for 24" floor slab at +40' | Case 4 |
| $C5 := C1_5$ | $C5 = 0.056605$ | Constant for 48" floor slab at +32' | Case 5 |

**Solving for $\omega$:**

Solving for $\omega 1$: 24" roof slab Case 1

$$0.59\omega 1^2 - \omega 1 + 0.002993 \text{ solve}, \omega 1 \rightarrow \begin{pmatrix} .29983039978494768920\text{e-}2 \\ 1.6919169502394386587 \end{pmatrix} \qquad \boxed{\omega 1r := 0.002993}$$

Solving for $\omega 2$: 24" roof slab Case 2

$$0.59\omega 2^2 - \omega 2 + 0.002627 \text{ solve}, \omega 2 \rightarrow \begin{pmatrix} .26310843368246154576\text{e-}2 \\ 1.6922841699004635201 \end{pmatrix} \qquad \boxed{\omega 2r := 0.002631}$$

Solving for $\omega 3$: 18" slab Case 3

$$0.59\omega 3^2 - \omega 3 + 0.004868 \text{ solve}, \omega 3 \rightarrow \begin{pmatrix} .48820623744867436938\text{e-}2 \\ 1.6900331918628013919 \end{pmatrix} \qquad \boxed{\omega 3r := 0.004882}$$

Solving for $\omega 4$: 24" slab Case 4

$$0.59\omega 4^2 - \omega 4 + 0.002920 \text{ solve}, \omega 4 \rightarrow \begin{pmatrix} .29250479843694094917\text{e-}2 \\ 1.6919902062529187261 \end{pmatrix} \qquad \boxed{\omega 4r := 0.002925}$$

Solving for $\omega 5$: 48" slab Case 5

$$0.59\omega 5^2 - \omega 5 + 0.056605 \text{ solve}, \omega 5 \rightarrow \begin{pmatrix} .58633342632256646839\text{e-}1 \\ 1.6362819116050314888 \end{pmatrix} \qquad \boxed{\omega 5r := 0.058633}$$

The quadratic equation calculates two values of $\omega$. (a) one value results in the percentage of steel that is more than (0.75 * pb) and therefore not used. (b) the second value of $\omega$ calculates the percentage of steel less than (0.75 * pb), and has been used in calculating the slab steel for out-of-plane bending moments (ACI 349-01, Ref. 2.2.3)

Therefore

$$\omega := \begin{pmatrix} \omega 1r \\ \omega 2r \\ \omega 3r \\ \omega 4r \\ \omega 5r \end{pmatrix} \qquad \omega = \begin{pmatrix} 0.00299 \\ 0.00263 \\ 0.00488 \\ 0.00293 \\ 0.05863 \end{pmatrix} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{array}$$

and

Required Reinforcing Ratio:

$$\boxed{\rho_{req} := \frac{\omega \cdot f_c}{f_y}}$$

Out-of-Plane (Vertical),

| | | |
|---|---|---|
| Case 1 | 24" roof slab at +100' | |
| Case 2 | 24" roof slab at +80' | |
| Case 3 | 18" floor slab at +40' | |
| Case 4 | 24" floor slab at +40' | |
| Case 5 | 48" floor slab at +32' | |

$$\boxed{\rho_{req} = \begin{pmatrix} 0.00025 \\ 0.00022 \\ 0.00041 \\ 0.00024 \\ 0.00489 \end{pmatrix}}$$

Note: For reinforcement summary- see Section 6.5.6 of this calculation for the total steel ( i.e due to Out-of-Plane (Vertical) and In-Plane (Horizontal) seismic acceleration loads). Minimum reinforcing steel for Out-of-Plane (Vertical) loading will be satisfied with total reinforcing as shown in Section 6.5.6 .

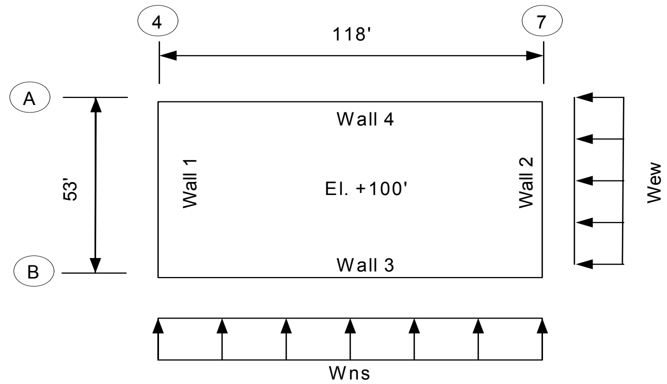
## 6.4 SLAB DIAPHRAGM DESIGN FOR IN-PLANE ( HORIZONTAL ) LOADS :

### 6.4.1 Diaphragm Analysis- Cases 1 to 5 :

Reinforcement steel for diaphragm considerations will be computed for the following diaphragm panels in this section:

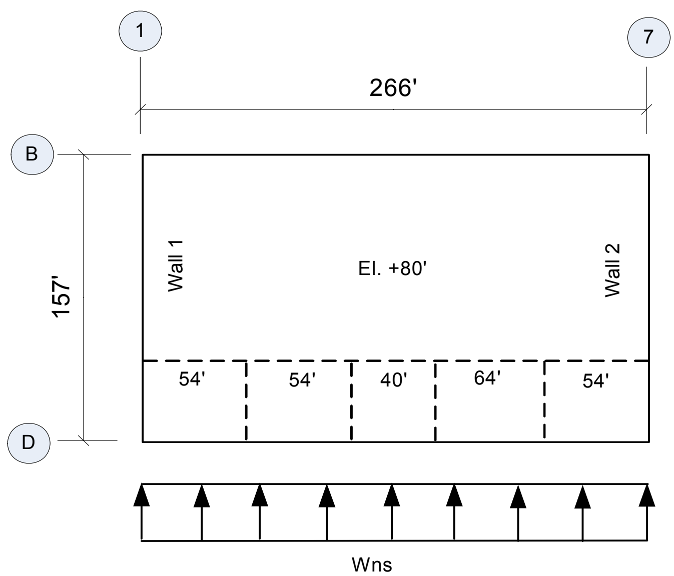
(See Plant Design Dwgs for Plans & Sections of WHF (Ref: 2.2.9 to 2.2.12), Attachment A & B )

| | | |
|---|---|---|
| Case 1: Roof Diaphragm @ +100' | Panel A - B / 4 - 7 ( N/S & E/W ) | 24" |
| Case 2: Roof Diaphragm @ + 80' | (a) Panel B - D / 1 - 7 ( N/S ) | 24" |
| | (b) Panel A - D / 1 - 7 ( E/W) | 24" |
| | (c) Panel A - B / 2 - 3 ( N/S ) (See Sect. 6.6) | 24" |
| Case 3: Slab Diaphragm @ + 40' | Panel B - C / 1 - 2 ( N/S & E/W ) | 18" |
| Case 4: Slab Diaphragm @ + 40' | Panel C - D / 4 - 6 ( N/S ) | 24" |
| | Panel A - B / 1 - 4 ( E/W) | 24" |
| Case 5: Slab Diaphragm @ + 32' | Panel A - B / 6 - 7 ( N/S ) | |
| | Panel A - B / 4 - 7 ( E/W ) | 48" |




NORTH

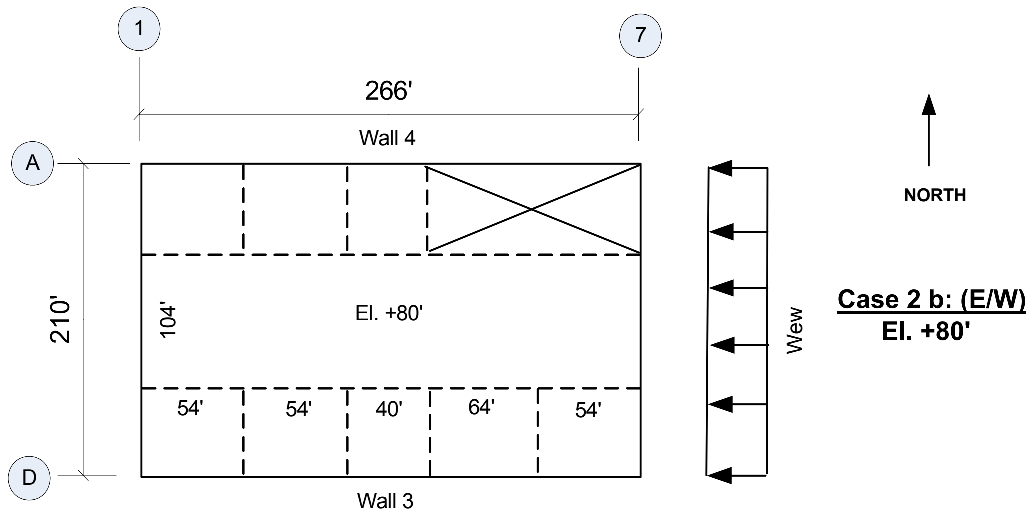
**<u>Case 1 : ( N/S & E/W )</u>**
**El. +100'**




NORTH

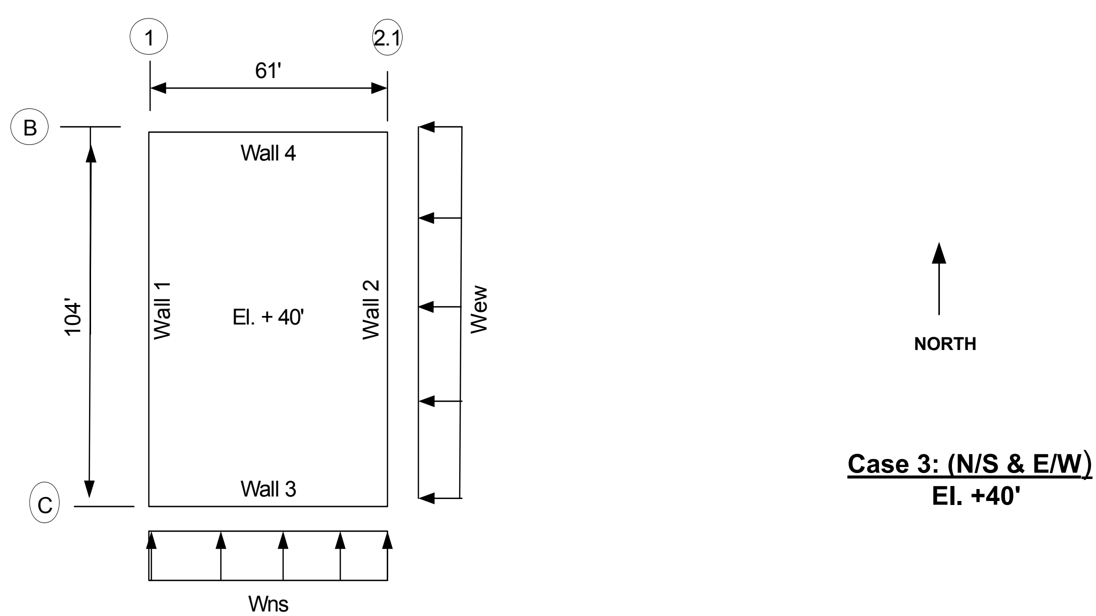
**<u>Case 2 a : (N/S)</u>**
**El. +80'**

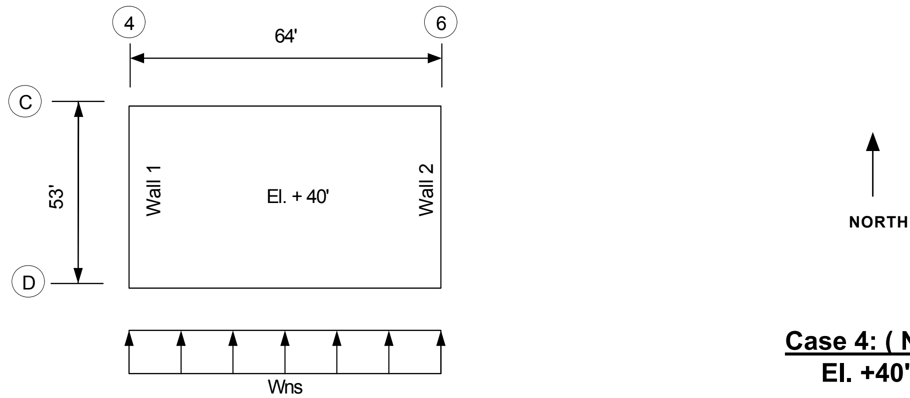
Note: For Case 2 a in the N/S direction, the diaphragm is taken as spanning from Column line 1 to 7 (266') with a depth of 157'. Since the internal shear walls provide support for the diaphragm, the largest spacing between walls will be taken as a 64' (between Column line 4 to 6) simple beam to calculate in-plane bending moments. This moment will be used to determine the diaphragm chord reinforcement from Column line 1 to 7. ( See Assumption 3.1.10 )



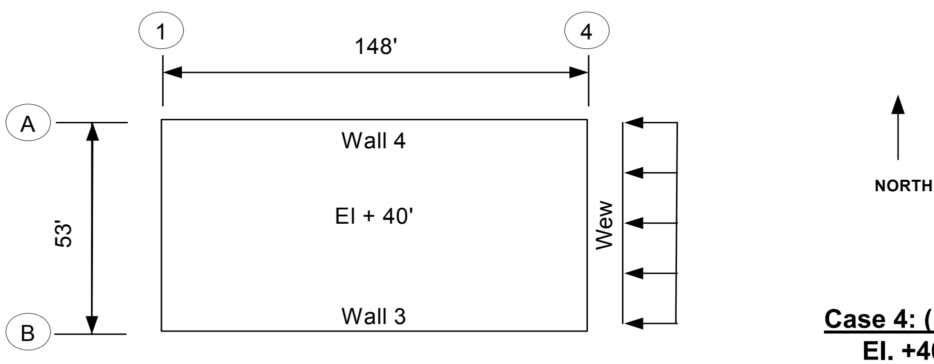

Note : For Case 2 b in the E/W direction, the diaphragm is a three spans system (see Attachment B). Conservatively, take diaphragm as simple span using the largest span ( that is, L = 104' between column lines B and C) ( see Assumption 3.2.2 )

For Case 2c, see Section 6.6

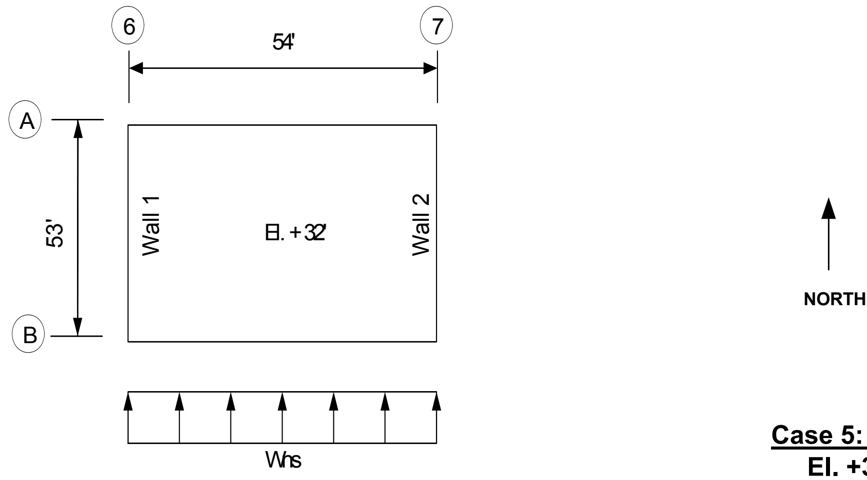




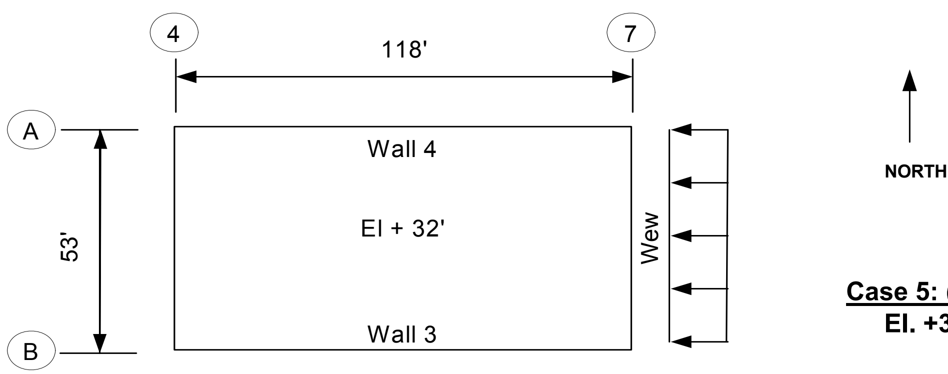

**Case 4: ( N/S)**
**El. +40'**




**Case 4: (E/W)**
**El. +40'**




**Case 5: (N/S)**
**El. +32'**




**Case 5: (E/W)**
**El. +32'**

### 6.4.2 Weight of Walls (per Foot) Tributary to Diaphragm : WWew

Note: Wall 1 is a **West wall** and Wall 2 is a **East wall**
Exterior walls located on periphery of the panel under consideration

#### 6.4.2.1 Weight of East & West Exterior Walls (per Foot) : $WW_{ew\_ext}$

(a) Tributary Height of Wall 1 (**West wall**) :( see Attachment A & B )

Case 1 : $Hw1\_1w := \frac{100 \cdot ft - 80 \cdot ft}{2}$ $Hw1\_1w = 10\,ft$

Col line 4 / A - B
Attachment A, Page A7
Attachment B, Page B2

Case 2 : $Hw1\_2w := \frac{80 \cdot ft - 40ft}{2}$ $Hw1\_2w = 20\,ft$

Col line 1 / A - D
Attachment A, Page A6
Attachment B, Page B2

Case 3 : $Hw1\_3w := \frac{80 \cdot ft - 40ft}{2} + \frac{40 \cdot ft - 0ft}{2}$ $Hw1\_3w = 40\,ft$

Col line 1 / B - C
Attachment A, Page A6
Attachment B, Page B3

Case 4 : $Hw1\_4w := \frac{80 \cdot ft - 40ft}{2} + \frac{40 \cdot ft - 0ft}{2}$ $Hw1\_4w = 40\,ft$

Col line 4 / C - D
Attachment A, Page A8
Attachment B, Page B3

Case 5 : $Hw1\_5w := \frac{32 \cdot ft - 0ft}{2}$ $Hw1\_5w = 16.00000\,ft$

Col line 6 / A - B
Attachment A, Page A7
Attachment B, Page B3

Thickness of Wall 1 :

$$Hw1w := \begin{pmatrix} Hw1\_1w \\ Hw1\_2w \\ Hw1\_3w \\ Hw1\_4w \\ Hw1\_5w \end{pmatrix} \qquad \boxed{Hw1w = \begin{pmatrix} 10 \\ 20 \\ 40 \\ 40 \\ 16 \end{pmatrix} ft}$$

Case 1 : Col line 4 / A-B
Case 2 : Col line 1 / A-D
Case 3 : Col line 1 / B-C
Case 4 : Col line 4 / C-D
Case 5 : Col line 4 / A-B

$$Tw1w := \begin{pmatrix} 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \end{pmatrix} \cdot ft$$

(b) Tributary Height of Wall 2 (**East wall**):( see Attachment A & B )

Case 1 : $Hw2\_1e := \frac{100 \cdot ft - 32 \cdot ft}{2}$ $Hw2\_1e = 34\,ft$

Col line 7 / A - B
Attachment A, Page A7
Attachment B, Page B2

Case 2 : $Hw2\_2e := \frac{80 \cdot ft - 0 \cdot ft}{2}$ $Hw2\_2e = 40\,ft$

Col line 7 / B - D
Attachment A, Page A6
Attachment B, Page B2

Case 3 : $Hw2\_3e := \frac{40 \cdot ft - 0 \cdot ft}{2}$ $Hw2\_3e = 20\,ft$

Col line 2.1 / B - C
Attachment A, Page A6
Attachment B, Page B3

Case 4 : $Hw2\_4e := \frac{80 \cdot ft - 40ft}{2} + \frac{40 \cdot ft - 0ft}{2}$ $Hw2\_4e = 40\,ft$

Col line 6 / C - D
Attachment A, Page A8
Attachment B, Page B3

Case 5 : $Hw2\_5e := \frac{100 \cdot ft - 32ft}{2} + \frac{32 \cdot ft - 0ft}{2}$ $Hw2\_5e = 50.00000\,ft$

Col line 7 / A - B
Attachment A, Page A7
Attachment B, Page B3

Thickness of Wall 2 :

$$Hw2e := \begin{pmatrix} Hw2\_1e \\ Hw2\_2e \\ Hw2\_3e \\ Hw2\_4e \\ Hw2\_5e \end{pmatrix} \qquad \boxed{Hw2e = \begin{pmatrix} 34 \\ 40 \\ 20 \\ 40 \\ 50 \end{pmatrix} ft}$$

Case 1 : Col line 7 / A - B
Case 2 : Col line 7 / A - D
Case 3 : Col line 2.1 / B-C
Case 4 : Col line 6 / C-D
Case 5 : Col line 7 / A-B

$$Tw2e := \begin{pmatrix} 4.0 \\ 4.0 \\ 2.0 \\ 4.0 \\ 4.0 \end{pmatrix} \cdot ft$$

$$WWew\_ext\_1\_5 := \overrightarrow{\left[(Hw1w \cdot Tw1w + Hw2e \cdot Tw2e) \cdot w_{conc}\right]}$$

Weight of East/West Exterior Walls (per foot)
Tributary to Diaphragm

$$WWew\_ext\_1\_5 = \begin{pmatrix} 26.4 \\ 36.0 \\ 30.0 \\ 48.0 \\ 39.6 \end{pmatrix} klf$$

Case 1
Case 2
Case 3
Case 4
Case 5

### 6.4.2.2 Weight Interior Walls (per Foot) : WWew_int

Diaphragm Spans for East/West Seismic Acceleration:

$$S_{ew1\_5} := \begin{pmatrix} 53 \\ 210 \\ 104 \\ 53 \\ 53 \end{pmatrix} \cdot ft$$

Case 1
Case 2
Case 3
Case 4
Case 5

Weights of interior walls normal to E/W acceleration :( see Attachment A & B )

Case 1 : No interior wall $WWew\_int\_1 := 0.0 \cdot kip$ ( see Attachment B, Page B2 )

Case 2 : (a) Interior six (6) walls at col line 2/A-B, 3/ A-B, 2/C-D, 3/C-D, 4/C-D & 6/C-D:
( see Attachment B)

$$H2a := \frac{80 \cdot ft - 40 \cdot ft}{2}$$

$H2a = 20\ ft$ Height

$T2a := 4 \cdot ft$ Thickness

$L2a := 53.ft$ Length

$$WWew\_int\_2a := \frac{H2a \cdot T2a \cdot L2a \cdot w_{conc}}{S_{ew1\_5_2}} \cdot 6$$

$WWew\_int\_2a = 18.2\,klf$

(b) Interior wall at col line 4/A-B : ( see Attachment B, Page B2 )

$$H2b := \frac{80 \cdot ft - 40 \cdot ft}{2} + \frac{100.ft - 80.ft}{2}$$

$H2b = 30.00000\ ft$ Height

$T2b := 4 \cdot ft$ Thickness

$L2b := 53 \cdot ft$ Length

$$WWew\_int\_2b := \frac{H2b \cdot T2b \cdot L2b \cdot w_{conc}}{S_{ew1\_5_2}}$$

$WWew\_int\_2b = 4.5\,klf$

Total =

$$WWew\_int\_2 := WWew\_int\_2a + WWew\_int\_2b$$

$WWew\_int\_2 = 22.7\,klf$

Case 3 : No interior wall $WWew\_int\_3 := 0.0 \cdot kip$

Case 4 : Interior two (2) E/W walls at col line 2/A-B and 3/A-B for panel A - B / 1 -4 : ( see Attachment B, Page B3)

$$H4 := \frac{80 \cdot ft - 40 \cdot ft}{2} + \frac{40.ft - 0.ft}{2} \qquad H4 = 40.00000\,ft \quad \text{Height}$$

$T4 := 4 \cdot ft$ Thickness

$L4 := 53 \cdot ft$ Length

$$WWew\_int\_4 := \frac{H4 \cdot T4 \cdot L4 \cdot w_{conc}}{S_{ew1\_5_4}} \cdot 2 \qquad WWew\_int\_4 = 48.0\,klf$$

Case 5 : Interior two (2) E/W walls @ col lines 5/A-B and 6/A-B for panel A-B / 4 - 7 ( see Attachment B, Page B3 )

$$H5 := \frac{32 \cdot ft - 0 \cdot ft}{2} \qquad H5 = 16.00000\,ft \quad \text{Height}$$

$T5 := 4 \cdot ft$ Thickness

$L5 := 53 \cdot ft$ Length

$$WWew\_int\_5 := \frac{H5 \cdot T5 \cdot L5 \cdot w_{conc}}{S_{ew1\_5_5}} \cdot 2 \qquad WWew\_int\_5 = 19.2\,klf$$

$$WWew\_int\_1\_5 := \begin{pmatrix} WWew\_int\_1 \\ WWew\_int\_2 \\ WWew\_int\_3 \\ WWew\_int\_4 \\ WWew\_int\_5 \end{pmatrix} \qquad WWew\_int\_1\_5 = \begin{pmatrix} 0.0 \\ 22.7 \\ 0.0 \\ 48.0 \\ 19.2 \end{pmatrix} klf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

(i.e.Weight of Interior Walls (per foot) Tributary to Diaphragm)

**6.4.2.3 Weight of Walls (per Foot) Tributary to Diaphragm : WWew**

$$\boxed{WWew1\_5 := WWew\_ext\_1\_5 + WWew\_int\_1\_5} \qquad WWew1\_5 = \begin{pmatrix} 26.4 \\ 58.7 \\ 30.0 \\ 96.0 \\ 58.8 \end{pmatrix} klf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

======

### 6.4.3 Weight of Walls (per Foot) Tributary to Diaphragm : WWns

Note: Wall 3 is a **South wall** and Wall 4 is a **North wall**
Exterior walls located on peripheri of the panel under consideration.

#### 6.4.3.1 Weight of North & South Exterior Walls (per Foot) : $WW_{ns\_ext}$

(a) Tributary Height of Wall 3 (**South wall**) :( see Attachment A & B )

Case 1 : $Hw3\_1s := \frac{100\cdot ft - 80\cdot ft}{2}$ $\quad Hw3\_1s = 10\,ft$

Col line B / 4 - 7
Attachment A, Page A2
Attachment B, Page B2

Case 2 : $Hw3\_2s := \frac{80\cdot ft - 40ft}{2}$ $\quad Hw3\_2s = 20\,ft$

Col line D / 1 - 7
Attachment A, Page A8
Attachment B, Page B2

Case 3 : $Hw3\_3s := \frac{80\cdot ft - 40ft}{2} + \frac{40\cdot ft - 0ft}{2}$ $\quad Hw3\_3s = 40\,ft$

Col line C / 1 - 2.1
Attachment A, Page A6
Attachment B, Page B3

Case 4 : $Hw3\_4s := \frac{80\cdot ft - 40ft}{2} + \frac{40\cdot ft - 0ft}{2}$ $\quad Hw3\_4s = 40\,ft$

Col line D / 4 -6
Attachment A, Page A8
Attachment B, Page B3

Case 5 : $Hw3\_5s := \frac{80\cdot ft - 32ft}{2} + \frac{32\cdot ft - 0ft}{2}$ $\quad Hw3\_5s = 40.00000\,ft$

Col line B / 6 -7
Attachment A, Page A7
Attachment B, Page B3

Thickness of Wall 3 :

$$Hw3s := \begin{pmatrix} Hw3\_1s \\ Hw3\_2s \\ Hw3\_3s \\ Hw3\_4s \\ Hw3\_5s \end{pmatrix} \quad Hw3s = \begin{pmatrix} 10 \\ 20 \\ 40 \\ 40 \\ 40 \end{pmatrix} ft$$

Case 1 : Col line B / 4-7
Case 2 : Col line D / 1-7
Case 3 : Col line C / 1-2.1
Case 4 : Col line D / 4-6
Case 5 : Col line B / 6-7

$$Tw3s := \begin{pmatrix} 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \end{pmatrix} \cdot ft$$

(b) Tributary Height of Wall 4 (**North wall**):( see Attachment A & B )

Case 1 : $Hw4\_1n := \frac{100 \cdot ft - 32 \cdot ft}{2}$ $Hw4\_1n = 34\,ft$

Col line A / 4 - 7
Attachment A, Page A2
Attachment B, Page B2

Case 2 : $Hw4\_2n := \frac{90 \cdot ft - 40 \cdot ft}{2}$ $Hw4\_2n = 25\,ft$

Col line B / 1 - 7
Attachment A, Page A8
Attachment B, Page B2

( Note: An equivalent area for the tributory wall at the 80' level was computed to obtain a uniform top of wall as elev. 90' for the wall between Column line B from 1 - 7)

Case 3 : $Hw4\_3n := \frac{80.ft - 40ft}{2} + \frac{40 \cdot ft - 0 \cdot ft}{2}$ $Hw4\_3n = 40\,ft$

Col line B / 1 - 2.1
Attachment A, Page A6
Attachment B, Page B3

Case 4 : $Hw4\_4n := \frac{80 \cdot ft - 40ft}{2} + \frac{40 \cdot ft - 0ft}{2}$ $Hw4\_4n = 40\,ft$

Col line C / 4 -6
Attachment A, Page A8
Attachment B, Page B3

Case 5 : $Hw4\_5n := \frac{100 \cdot ft - 32ft}{2} + \frac{32 \cdot ft - 0ft}{2}$ $Hw4\_5n = 50.00000\,ft$

Col line A / 6 -7
Attachment A, Page A7
Attachment B, Page B3

Thickness of Wall 4 :

$$Hw4n := \begin{pmatrix} Hw4\_1n \\ Hw4\_2n \\ Hw4\_3n \\ Hw4\_4n \\ Hw4\_5n \end{pmatrix} \qquad \boxed{Hw4n = \begin{pmatrix} 34 \\ 25 \\ 40 \\ 40 \\ 50 \end{pmatrix} ft}$$

Case 1 : Col line A / 4-7
Case 2 : Col line B / 1-7
Case 3 : Col line B / 1-2.1
Case 4 : Col line C / 4-6
Case 5 : Col line A / 6-7

$$Tw4n := \begin{pmatrix} 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \\ 4.0 \end{pmatrix} \cdot ft$$

$$WWns\_ext\_1\_5 := \overrightarrow{\left[(Hw3s \cdot Tw3s + Hw4n \cdot Tw4n) \cdot w_{conc}\right]}$$

( i.e. Weight of North/South Exterior Walls (per foot) Tributary to Diaphragm for all 5 cases)

$$WWns\_ext\_1\_5 = \begin{pmatrix} 26.4 \\ 27.0 \\ 48.0 \\ 48.0 \\ 54.0 \end{pmatrix} klf \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

**6.4.3.2 Weight of <u>Interior Walls</u> (per Foot) : WW**ns_int:

Diaphragm Spans ( For North/South Seismic Acceleration)

$$S_{ns1\_5} := \begin{pmatrix} 118 \\ 266 \\ 61 \\ 64 \\ 54 \end{pmatrix} \cdot ft \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Weights of all interior walls:<u>( see Attachment A & B )</u>

<u>Case 1 : No interior wall</u>

$WWns\_int\_1ns := 0.0 \cdot kip$

<u>Case 2 :</u> (a) Interior wall @ col line C / 1 - 7 (see Attachment B)

$$H2ans := \frac{80 \cdot ft - 40 \cdot ft}{2}$$

$H2ans = 20\ ft$ Height

$T2ans := 4 \cdot ft$ Thickness

$L2ans := 266.ft$ Length

$$WWns\_int\_2ans := \frac{H2ans \cdot T2ans \cdot L2ans \cdot w_{conc}}{S_{ns1\_5_2}}$$

$WWns\_int\_2ans = 12.00000\,kl$

Total = $WWns\_int\_2ns := WWns\_int\_2ans$

$WWns\_int\_2ns = 12.0\,klf$

<u>Case 3 : No interior wall</u>

$WWns\_int\_3ns := 0.0 \cdot kip$

<u>Case 4 : No interior wall</u>

$WWns\_int\_4ns := 0.0 \cdot kip$

<u>Case 5 : No interior wall</u>

$WWns\_int\_5ns := 0.0 \cdot kip$

$$WWns\_int\_1\_5 := \begin{pmatrix} WWns\_int\_1ns \\ WWns\_int\_2ns \\ WWns\_int\_3ns \\ WWns\_int\_4ns \\ WWns\_int\_5ns \end{pmatrix} \qquad WWns\_int\_1\_5 = \begin{pmatrix} 0.0 \\ 12.0 \\ 0.0 \\ 0.0 \\ 0.0 \end{pmatrix} klf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

(i.e.Weight of Interior Walls (per foot) Tributary to Diaphragm for 5 cases)

#### 6.4.3.3 Weight of North and South Walls (per Foot) Tributary to Diaphragm : WWns

$$WWns1\_5 := WWns\_ext\_1\_5 + WWns\_int\_1\_5 \qquad WWns1\_5 = \begin{pmatrix} 26.4 \\ 39.0 \\ 48.0 \\ 48.0 \\ 54.0 \end{pmatrix} klf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

### 6.4.4 Design of diaphragm for North/South Seismic Acceleration:

#### 6.4.4.1 Determine Design Loads ( For Horizontal Loads ):

Slab Thicknesses: (See Section 6.3.5)

$$h_1 := \begin{pmatrix} 24 \\ 24 \\ 18 \\ 24 \\ 48 \end{pmatrix} \cdot in \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Live Loads (See Section 6.2.10 )

$$LL_1 := \begin{pmatrix} 40 \\ 40 \\ 100 \\ 100 \\ 100 \end{pmatrix} \cdot Psf$$

and

Total Dead Load (TLD): (See Section 6.3.1)

$$TDL := \begin{pmatrix} TDL_{100} \\ TDL_{80} \\ TDL_{40a} \\ TDL_{40b} \\ TDL_{32} \end{pmatrix} \qquad TDL = \begin{pmatrix} 424 \\ 424 \\ 384 \\ 459 \\ 700 \end{pmatrix} Psf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Combine dead load and live load for seismic load consideration as follows:

The Design Loads ( For Horizontal Loads )

$$\boxed{U3 := TDL + 0.25 \cdot LL_1} \quad \text{Psf}$$

$$U3 = \begin{pmatrix} 434 \\ 434 \\ 409 \\ 484 \\ 725 \end{pmatrix} \text{Psf} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

**6.4.4.2 Moments and Shears :**

$$Depth_1 := \begin{pmatrix} 53 \\ 157 \\ 104 \\ 53 \\ 53 \end{pmatrix} \cdot ft \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

N-S Loading Diaphragm Depths

$$SACC_{y1} := \begin{pmatrix} a100_y \\ a80_y \\ a40_y \\ a40_y \\ a32_y \end{pmatrix} \qquad \boxed{SACC_{y1} = \begin{pmatrix} 1.63 \\ 0.98 \\ 0.74 \\ 0.74 \\ 0.69 \end{pmatrix}} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Seismic Accelerations in Y (North/South) Direction

$$\boxed{W_{ns1} := \overrightarrow{\left[\left(U3 \cdot Depth_1 + WWns1\_5\right) \cdot SACC_{y1}\right]}}$$

i.e. Horizontal Seismic Load per foot (N/S)

$$W_{ns1} = \begin{pmatrix} 80.4 \\ 104.7 \\ 67.3 \\ 54.8 \\ 64.0 \end{pmatrix} \text{kft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Loads

Note: The Canister Transfer Machine (CTM) supported by walls A and b will impose a 155.4 kip load at Elevation 100 ft. (See Attachment C, Ref. 2.2.18). Using the 118 ft. length of diaphragm (Case 1), the 155.4 kip load becomes a uniform load of 1.32 k/lf. This uniform load is less than 3% of the total uniform load used for the north/south acceleration for Case 1. This small load is not considered significant for this preliminary design and is not included. Similar analyses and conclusions can be made for other diaphragms with induced loads from the 200-ton crane and the CTM. See Assumption 3.1.11.

$$Span_1 := \begin{pmatrix} 118 \\ 64 \\ 61 \\ 64 \\ 54 \end{pmatrix} \cdot ft$$

Case 1
Case 2
Case 3
Case 4
Case 5

Diaphragm Spans

$$M_{ns1} := \overrightarrow{\frac{W_{ns1} \cdot Span_1^{\,2}}{8}}$$

$$M_{ns1} = \begin{pmatrix} 139859 \\ 53628 \\ 31321 \\ 28051 \\ 23313 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4
Case 5

Moments

$$V_{ns1} := \overrightarrow{\frac{W_{ns1} \cdot Span_1}{2}}$$

$$V_{ns1} = \begin{pmatrix} 4741 \\ 3352 \\ 2054 \\ 1753 \\ 1727 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4
Case 5

Shears

**6.4.4.3 Check Nominal Shear Capacity of Concrete per Code ACI 349 ( Ref: 2.2.3, Section 21.6.5.6 )** :

$$h_1 = \begin{pmatrix} 24 \\ 24 \\ 18 \\ 24 \\ 48 \end{pmatrix} in$$

Case 1
Case 2
Case 3
Case 4
Case 5

Slab thicknesses from Section 6.3.5

$$A_{cv1} := \overrightarrow{\left(Depth_1 \cdot h_1\right)} \qquad A_{cv1} = \begin{pmatrix} 106 \\ 314 \\ 156 \\ 106 \\ 212 \end{pmatrix} ft^2 \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix} \quad \text{Shear Areas}$$

$\phi_{diaph} := 0.60$ (strength reduction factor for in-plane shear per ACI 349 ( Ref: 2.2.3, Section 9.3.4 )

$$\phi V_{n\_max1} := \phi_{diaph} \cdot 8 \cdot A_{cv1} \cdot \sqrt{f_c \cdot psi} \qquad \phi V_{n\_max1} = \begin{pmatrix} 5181 \\ 15347 \\ 7625 \\ 5181 \\ 10362 \end{pmatrix} kip \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix} \quad \text{Concrete Shear Capacity}$$

(Ref: 2.2.3, Section 21.6.5.6)

$\phi V_{n\_max1} > V_{ns1}$ **Therefore, the limiting shear capacity satisfied.**

**6.4.4.4 Check Concrete Shear Capacity per Code ACI 349 ( Ref: 2.2.3, Section 21.6.5.2 ):**

Vn = Nominal Shear Strength = (Vc + Vs) where

$V_c$ Concrete Shear Capacity provided by Concrete &

Vs Nominal Shear Strength provided by Shear Reinforcement

$V_{n1} = V_{c1} + V_{s1}$ ( Ref: 2.2.3, Section 11.1 )

$$\frac{V_{ns1}}{\phi_{diaph}} := V_{c1} + V_{s1}$$

where, Vns1 = Actual Shear Capacity due to N/S Loading

or $\phi_{diaph} \cdot V_{c1} = [\ Vns1 - \phi_{diaph} \cdot V_{s1}\ ]$ $\phi_{diaph} = 0.60$ fc = 5000 psi

From

$$V_{c1} := 2 \cdot A_{cv1} \cdot \sqrt{f_c \cdot psi} \qquad \phi_{diaph} \cdot V_{c1} = \begin{pmatrix} 1295 \\ 3837 \\ 1906 \\ 1295 \\ 2590 \end{pmatrix} kip \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix} \quad \text{Shear Capacity provided by Concrete}$$

(Ref: 2.2.3, Section 21.6.5.2)

Since $V_{ns1} > \phi_{diaph} \cdot V_{c1}$ **Shear Reinforcing is required for all the Cases 1, 2, 3 and 4**

$$V_{s1} := \frac{V_{ns1}}{\phi_{diaph}} - V_{c1}$$

$$V_{s1} = \begin{pmatrix} 5743 \\ -808 \\ 246 \\ 763 \\ -1439 \end{pmatrix} kip \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Nominal Shear Strength provided by Shear reinforcement.

Revising the negative values of

rewrite the above equation as -

$$V_{s1\_rev} := \left| \begin{array}{l} V_{s1} \ \text{ if } \ V_{s1_5} \geq 0.0kip \\ \begin{pmatrix} V_{s1_1} \\ V_{s1_2} \\ V_{s1_3} \\ V_{s1_4} \\ 0.0 \cdot kip \end{pmatrix} \end{array} \right.$$

$$V_{s1\_rev} = \begin{pmatrix} 5742.96822 \\ -808.23724 \\ 246.19885 \\ 763.33002 \\ 0.00000 \end{pmatrix} kip$$

$$V_{s1\_rev} := \left| \begin{array}{l} V_{s1} \ \text{ if } \ V_{s1_2} \geq 0.0kip \\ \begin{pmatrix} V_{s1_1} \\ 0.0kip \\ V_{s1_3} \\ V_{s1_4} \\ 0.0 \cdot kip \end{pmatrix} \end{array} \right.$$

$$V_{s1\_rev} = \begin{pmatrix} 5742.96822 \\ 0.00000 \\ 246.19885 \\ 763.33002 \\ 0.00000 \end{pmatrix} \text{kip}$$

#### 6.4.4.5 Required reinforcement for In-plane (Horizontal) Loads N/S Seismic acceleration:

From $Vn = Acv\ (\ 2 \times f'c^{1/2}\ +\ \rho_n \times f_y\ )$ (Ref. 2.2.3, Section 21.6.5.2)

we have

$$Vn = Acv \times 2 \times f'c^{1/2} + Acv \times \rho_n \times f_y$$

$$Vn = Vc + (Acv \times \rho_n \times f_y)$$

$$(Vn - Vc) = Acv \times \rho_n \times f_y$$

$$\rho_n = Vs\ /\ (\ Acv \times f_y\ )$$

$$\boxed{\rho_{req1} := \frac{V_{s1\_rev}}{A_{cv1} \cdot f_y}}$$

(Ref. 2.2.3, Section 21.6.5.2)

Ratio $\rho_{req1} = \begin{pmatrix} 0.00627 \\ 0.00000 \\ 0.00018 \\ 0.00083 \\ 0.00000 \end{pmatrix}$ Case 1, Case 2, Case 3, Case 4, Case 5

Shear Requirements (total of steel required on 2 faces)

Note : Use same top and bottom bars. See reinforcement summary for total steel required for out of plane (vertical) and in-plane (horizontal) loads.

### 6.4.5 Design of diaphragm for East/West Seismic Acceleration:

#### 6.4.5.1 Moments and Shears

$$\boxed{Depth_2 := \begin{pmatrix} 118 \\ 266 \\ 61 \\ 148 \\ 118 \end{pmatrix} \cdot ft}$$

Case 1, Case 2, Case 3, Case 4, Case 5

Diaphragm Depths

$$SACC_{x1} := \begin{pmatrix} a100_x \\ a80_x \\ a40_x \\ a40_x \\ a32_x \end{pmatrix}$$

$$\boxed{SACC_{x1} = \begin{pmatrix} 1.34 \\ 0.99 \\ 0.73 \\ 0.73 \\ 0.74 \end{pmatrix}}$$

Case 1
Case 2
Case 3
Case 4
Case 5

Seismic Accelerations in X (East/West) Direction

$$W_{ew1} := \overrightarrow{\left[\left(U3 \cdot Depth_2 + WWew1\_5\right) \cdot SACC_{x1}\right]}$$

i.e. Horizontal Seismic Ld per foot (E/W dir.)

$$W_{ew1} = \begin{pmatrix} 104 \\ 172 \\ 40 \\ 123 \\ 107 \end{pmatrix} kft$$

Case 1
Case 2
Case 3
Case 4
Case 5

Loads

Note: Case 2 - For the E/W loading, the longest span is taken as 104' (=BC) instead of 210' (=AD) for design purpose.

$$Span_2 := \begin{pmatrix} 53 \\ 104 \\ 104 \\ 53 \\ 53 \end{pmatrix} \cdot ft$$

Case 1
Case 2
Case 3
Case 4
Case 5

Diaphragm Spans

$$M_{ew1} := \overrightarrow{\frac{W_{ew1} \cdot Span_2^{\,2}}{8}}$$

$$M_{ew1} = \begin{pmatrix} 36503 \\ 232312 \\ 54440 \\ 43135 \\ 37558 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4
Case 5

Moments

$$V_{ew1} := \overrightarrow{\frac{W_{ew1} \cdot Span_2}{2}}$$

$$V_{ew1} = \begin{pmatrix} 2755 \\ 8935 \\ 2094 \\ 3255 \\ 2835 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4
Case 5

Shears

**6.4.5.2 Check Concrete Nominal Shear Capacity per Code ACI 349 ( Ref: 2.2.3 ,**

**Section 21.6.5.6 )** :

$A_{cv2} := \overrightarrow{\left(Depth_2 \cdot h_1\right)}$

$A_{cv2} = \begin{pmatrix} 236 \\ 532 \\ 91 \\ 296 \\ 472 \end{pmatrix} ft^2$ Case 1, Case 2, Case 3, Case 4, Case 5 — Shear Areas

$\phi_{diaph} = 0.60$ strength reduction factor for in-plane shear per ACI 349 ( Ref: 2.2.3, Sect. 9.3.4 )

$\boxed{\phi V_{n\_max2} := \phi_{diaph} \cdot 8 \cdot A_{cv2} \cdot \sqrt{f_c \cdot psi}}$

(Ref: 2.2.3, Sect. 21.6.5.6)

$\phi V_{n\_max2} = \begin{pmatrix} 11535 \\ 26002 \\ 4472 \\ 14467 \\ 23069 \end{pmatrix} kip$ Case 1, Case 2, Case 3, Case 4, Case 5 — Concrete Shear Capacity

$\phi V_{n\_max2} > V_{ew1}$, **Therefore, the limiting diaphragm shear capacity is satisfied.**

**6.4.5.3 Check Concrete Shear Capacity per Code ACI 349 ( Ref: 2.2.3, Section 21.6.5.2 ):**

Vn = Nominal Shear Strength = (Vc + Vs) where

$V_c$ Concrete Shear Capacity provided by Concrete &

Vs Nominal Shear Strength provided by Shear Reinforcement

$V_{n2} := V_{c2} + V_{s2}$ ( Ref: 2.2.3, Section 11.1 )

$\frac{V_{ew1}}{\phi_{diaph}} := V_{c2} + V_{s2}$ where, Vns1 = Actual Shear Capacity due to N/S Loading

or $\phi_{diaph} \cdot V_{c2}$ = [ Vew2 - $\phi_{diaph} \cdot V_{s2}$ ] $\phi_{diaph} =$ 0.60 fc = 5000 psi

From

$\boxed{V_{c2} := 2 \cdot A_{cv2} \cdot \sqrt{f_c \cdot psi}}$

( Ref: 2.2.3, Sect. 21.6.5.2)

$\phi_{diaph} \cdot V_{c2} = \begin{pmatrix} 2884 \\ 6500 \\ 1118 \\ 3617 \\ 5767 \end{pmatrix} kip$ Case 1, Case 2, Case 3, Case 4, Case 5 — Concrete Shear Capacity provided by Concrete

Since $\boxed{\phi_{diaph} \cdot V_{c2} < V_{ew1}}$ Therefore, Shear Reinforcing is required for Cases 2 and 3

$$V_{s2} := \frac{V_{ew1}}{\phi_{diaph}} - V_{c2}$$

$$V_{s2} = \begin{pmatrix} -214 \\ 4058 \\ 1626 \\ -602 \\ -4888 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4
Case 5

Nominal Shear Strength provided by Shear reinf.

Revising the negative values of $V_{s2}$ rewrite the above equation as -

$$V_{s2\_rev} := \left| \begin{array}{l} V_{s2} \ \text{ if } \ V_{s2_5} \ge 0.0kip \\ \begin{pmatrix} V_{s2_1} \\ V_{s2_2} \\ V_{s2_3} \\ V_{s2_4} \\ 0.0 \cdot kip \end{pmatrix} \end{array} \right.$$

and

$$V_{s2\_rev} = \begin{pmatrix} -214 \\ 4058 \\ 1626 \\ -602 \\ 0 \end{pmatrix} kip$$

$$V_{s2\_rev} := \left| \begin{array}{l} V_{s2} \ \text{ if } \ V_{s2_1} \ge 0.0kip \\ \begin{pmatrix} 0.kip \\ V_{s2_2} \\ V_{s2_3} \\ V_{s2_4} \\ 0.0 \cdot kip \end{pmatrix} \end{array} \right.$$

$$V_{s2\_rev} = \begin{pmatrix} 0.00000 \\ 4057.80409 \\ 1626.39024 \\ -602.19543 \\ 0.00000 \end{pmatrix} kip$$

$$V_{s2\_rev} := \left| \begin{array}{l} V_{s2} \ \text{ if } \ V_{s2_4} \geq 0.0\text{kip} \\ \begin{pmatrix} 0.\text{kip} \\ V_{s2_2} \\ V_{s2_3} \\ 0.\text{kip} \\ 0 \cdot \text{kip} \end{pmatrix} \end{array} \right.$$

$$V_{s2\_rev} = \begin{pmatrix} 0.00000 \\ 4057.80409 \\ 1626.39024 \\ 0.00000 \\ 0.00000 \end{pmatrix} \text{kip}$$

### 6.4.5.4 Required reinforcement for In-plane (Horizontal) Loads E/W Seismic accel.:

$$\rho_{req2} := \frac{V_{s2\_rev}}{A_{cv2} \cdot f_y}$$ Ratio

(Ref. 2.2.3, Section 21.6.5.2 )

$$\rho_{req2} = \begin{pmatrix} 0.00000 \\ 0.00088 \\ 0.00206 \\ 0.00000 \\ 0.00000 \end{pmatrix} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{array}$$

Steel Requirements (total ratio of steel required on 2 faces)

Note : Use same top and bottom bars.
See Reinforcement Summary for total steel required and in-plane (horizontal) loads.

## 6.5 DETERMINE TOTAL REINFORCEMENT :

### 6.5.1 Diaphragm Locations: (From Sect. 6.4.1 of this calculation)

| | | |
|---|---|---|
| Case 1: Roof Diaphragm @ +100' | Panel A - B / 4 - 7 ( N/S & E/W ) | 24" |
| Case 2: Roof Diaphragm @ + 80' | (a) Panel B - D / 1 - 7 ( N/S ) | 24" |
| | (b) Panel A - D / 1 - 7 ( E/W) | 24" |

| | | |
|---|---|---|
| | (c) Panel A - B / 2 - 3 ( N/S ) (See Sect. 6.6) | 24" |
| Case 3: Slab Diaphragm @ + 40' | Panel B - C / 1 - 2 ( N/S & E/W ) | 18" |
| Case 4: Slab Diaphragm @ + 40' | Panel C - D / 4 - 6 ( N/S ) | 24" |
| | Panel A - B / 1 - 4 ( E/W) | 24" |
| Case 5: Slab Diaphragm @ + 32' | Panel A - B / 6 - 7 ( N/S ) | |
| | Panel A - B / 4 - 7 ( E/W ) | 48" |

### 6.5.2 Reinforcement for <u>Out-of-Plane (Vertical)</u> Loads : ( From Section 6.3.6 )

$$\rho_{req} = \begin{pmatrix} 0.00025 \\ 0.00022 \\ 0.00041 \\ 0.00024 \\ 0.00489 \end{pmatrix} \quad \begin{matrix} \text{24" roof slab} & \text{Case 1} \\ \text{24" roof slab} & \text{Case 2} \\ \text{18" floor slab} & \text{Case 3} \\ \text{24" roof slab} & \text{Case 4} \\ \text{48" floor slab} & \text{Case 5} \end{matrix}$$

### 6.5.3 Reinforcement for <u>In-Plane (Horizontal)</u> Loads : ( From Sections 6.4.4.5 & 6.4.5.4)

<u>N/S Seismic</u>

$$\rho_{req1} = \begin{pmatrix} 0.00627 \\ 0 \\ 0.00018 \\ 0.00083 \\ 0 \end{pmatrix} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

<u>E/W Seismic</u>

$$\rho_{req2} = \begin{pmatrix} 0.00000 \\ 0.00088 \\ 0.00206 \\ 0.00000 \\ 0.00000 \end{pmatrix} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

### 6.5.4 Total <u>Combined</u> Reinforcement required ( for In-plane & Out- of -plane Loads ):

$$\rho_{req\_comb} := \begin{pmatrix} \rho_{req_1} + \dfrac{\max\left(\rho_{req1_1}, \rho_{req2_1}\right)}{2} \\ \rho_{req_2} + \dfrac{\max\left(\rho_{req1_2}, \rho_{req2_2}\right)}{2} \\ \rho_{req_3} + \dfrac{\max\left(\rho_{req1_3}, \rho_{req2_3}\right)}{2} \\ \rho_{req_4} + \dfrac{\max\left(\rho_{req1_4}, \rho_{req2_4}\right)}{2} \\ \rho_{req_5} + \dfrac{\max\left(\rho_{req1_5}, \rho_{req2_5}\right)}{2} \end{pmatrix} \quad \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{array}$$

Therefore, steel ratio = $\boxed{\rho_{req\_comb} = \begin{pmatrix} 0.00338 \\ 0.00066 \\ 0.00144 \\ 0.00066 \\ 0.00489 \end{pmatrix}}$

For $d_{1\_5} := \begin{pmatrix} 21.13 \\ 21.13 \\ 15.13 \\ 21.13 \\ 45.13 \end{pmatrix} \cdot in \quad \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{array}$

$$As_{req1\_5} := \overrightarrow{\left[\left(\rho_{req\_comb}\right) \cdot b \cdot d_{1\_5}\right]} \qquad \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{array} \qquad \boxed{As_{req1\_5} = \begin{pmatrix} 0.85824 \\ 0.16752 \\ 0.26062 \\ 0.16747 \\ 2.64611 \end{pmatrix} in^2}$$

### 6.5.5 Check and determine Minimum Reinforcement Req'd. per Code:

ACI 349-02 ( Ref: 2.2.3 ), Section 21.6.2.1 specifies that the minimum reinforcement ratio for structural diaphragms shall be in conformance with Section 7.12.5

ACI 349-02 ( Ref: 2.2.3 ), Section 7.12.5 requires that where reinforcement is required the ratio of reinforcement provided on the tension face shall not be less than **.0018 times the gross concrete area.**

For $h_{1\_5} := \begin{pmatrix} 24 \\ 24 \\ 18 \\ 24 \\ 48 \end{pmatrix} \cdot in$ Case 1, Case 2, Case 3, Case 4, Case 5 and b = 12 in

$As_{req1\_5\_min} := 0.0018 \cdot b \cdot h_{1\_5}$

Case 1, Case 2, Case 3, Case 4, Case 5 $As_{req1\_5\_min} = \begin{pmatrix} 0.51840 \\ 0.51840 \\ 0.38880 \\ 0.51840 \\ 1.03680 \end{pmatrix} in^2$

Use maximum values from A and B above as follows:

$$As_{reqd1\_5} := \begin{pmatrix} \max\left(As_{req1\_5_1}, As_{req1\_5\_min_1}\right) \\ \max\left(As_{req1\_5_2}, As_{req1\_5\_min_2}\right) \\ \max\left(As_{req1\_5_3}, As_{req1\_5\_min_3}\right) \\ \max\left(As_{req1\_5_4}, As_{req1\_5\_min_4}\right) \\ \max\left(As_{req1\_5_5}, As_{req1\_5\_min_5}\right) \end{pmatrix}$$

Case 1, Case 2, Case 3, Case 4, Case 5

The required design reinforcements for diaphrams

$As_{reqd1\_5} = \begin{pmatrix} 0.85824 \\ 0.51840 \\ 0.38880 \\ 0.51840 \\ 2.64611 \end{pmatrix} in^2$

Case 1
Case 2
Case 3 per foot
Case 4
Case 5

### 6.5.6 Diaphragm Reinforcement:

**Provide Reinforcement for slabs / diaphrams as follows: (SUMMARY)**

Case 1: Provide # 10 bars @ 12" on center, both ways, top & bottom ( As = 1.27 $in^2$ )

Case 2: Provide # 8 bars @ 12" on center, both ways, top & bottom ( As = 0.79 $in^2$ ).

Case 3 : Provide # 8 bars @ 12" on center, both ways, top & bottom (As = 0.79 $in^2$ )

Case 4 : Provide # 8 bars @ 12" on center, both ways, top & bottom (As = 0.79 $in^2$ )

Case 5 : Provide # 11 bars @ 6" on center, both ways, top & bottom (As = 3.12 $in^2$ )

and for Slab at +20' elevation, Panel B - C / 1 -2 ( same reinforcement as Case 3 above) :
Provide # 8 bars @ 12" on center, both ways, top & bottom (As = 0.79 $in^2$ )

Reinf. Steel Provided as,

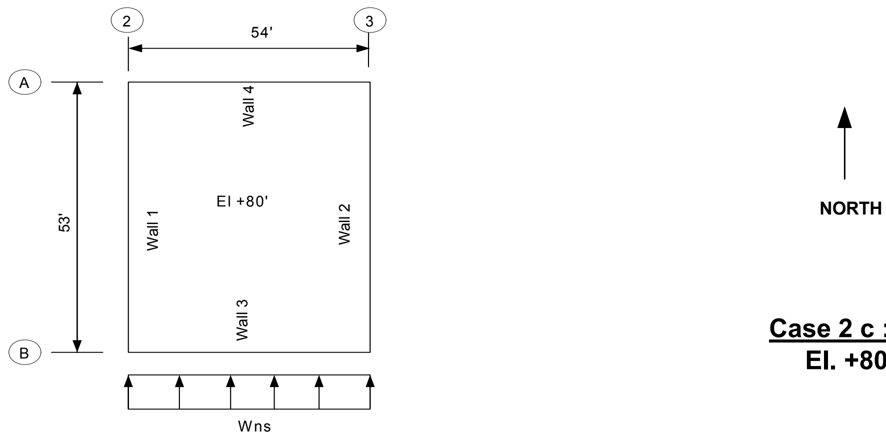
$$As_{prov1\_5} := \begin{pmatrix} 1.27 \\ 0.79 \\ 0.79 \\ 0.79 \\ 3.12 \end{pmatrix} \cdot in^2 \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix} \quad \text{per foot}$$

From the above, Reinforcement provided for all Cases 1 to 5 is more than required in each case

## 6.6 Weight of North and South Walls (per Foot) Tributary to Diaphragm : WWns2c

**Case 2c Panel A-B/2-3 Chord Reinforcement for N-S Loading**

Note: Wall 3 is a **South wall** and Wall 4 is a **North wall**




**Case 2 c : (N/S)**
**El. +80'**

### 6.6.1 Weight of Walls (per Foot) : $WW_{ns\_ext}$

Tributary Height of Wall 3 (**South wall**) :

$$Hw3\_2sc := \frac{90\cdot ft - 40ft}{2}$$

$\boxed{Hw3\_2sc = 25ft}$

Case 2a : Col line B / 2 - 3

(Note: See Section 6.4.3.1, (b)
Case 2 North wall)

$Tw3sc := 4.ft$

Tributary Height of Wall 4 (**North wall**):

$$Hw4\_2nc := \frac{90\cdot ft - 40\cdot ft}{2}$$

$\boxed{Hw4\_2nc = 25ft}$

Case 2c : Col line A / 2-3

(Note: See Section 6.4.3.1, (b)
Case 2 North wall)

$Tw4nc := 4.ft$

$$WWns\_ext\_2c := \overrightarrow{\left[(Hw3\_2sc\cdot Tw3sc + Hw4\_2nc\cdot Tw4nc)\cdot w_{conc}\right]}$$

$WWns\_ext\_2c = 30.00000\,klf$

Weight of North/South Exterior Walls (per foot)
Tributary to Diaphragm

$\boxed{S_{ns2c} := 54\cdot ft}$ Diaphragm Spans ( For North/South Seismic Acceleration)

### 6.6.2 Design of diaphragm for North/South Seismic Acceleration:

#### 6.6.2.1 Determine Design Loads ( For Horizontal Loads ):

Slab Thicknesses: (See Sect.6.3.5 of this calc) $h_1 := 24 \cdot in$

Live Loads (See Section 6.2.10) $LL_1 := 40 \cdot Psf$

and

Total Dead Load (TLD): (See Section 6.3.1) $TDL := TDL_{80}$ $TDL = 424\,Psf$

Combine dead load and live load for seismic load consideration as follows:

The Design Loads ( For Horizontal Loads ) $U3 := TDL + 0.25 \cdot LL_1$ Psf $U3 = 434\,Psf$

#### 6.6.2.2 Moments :

$Depth_{1c} := 53 \cdot ft$ Diaphragm Depths

$SACC_{y1} := a80_y$ $SACC_{y1} = 0.98$ Case 2c Seismic Accelerations in Y (North/South) Direction

$$W_{ns1c} := \overrightarrow{\left[\left(U3 \cdot Depth_{1c} + WWns\_ext\_2c\right) \cdot SACC_{y1}\right]}$$

i.e. Horizontal Seismic Load per foot (N/S dir.) $W_{ns1c} = 51.8\,kft$

$Span_{1c} := 54 \cdot ft$ Case 2c Diaphragm Span

$$M_{ns1c} := \overrightarrow{\frac{W_{ns1c} \cdot Span_{1c}^{2}}{8}}$$

$M_{ns1c} = 18889.48259\,ft \cdot kip$ Moment

### 6.6.2.3 Check Chord Steel Requirements: Panel A-B/2-4 (Case 2a)

**For North/South Seismic Acceleration:**

$\boxed{M_{ns1c} = 18889\,ft \cdot kip}$ $\boxed{Depth_{1c} = 53\ ft}$

Chord Force (CF): $CF_{1c} := \dfrac{M_{ns1c}}{0.90 \cdot Depth_{1c}}$ Case 2c $CF_{1c} = 396005.92429\ lbf$

Note : 0.90 x Depth1 is taken as lever arm from centroid of compression stress block to centroid of chord reinforcing steel. ( See attachment C and Assumption 3.2.3 )

Required Chord Steel (Ach):

$\phi_b := 0.90$ Capacity Reduction Factor for bending (Ref: 2.2.3, Section 9.3.2.1)

$\boxed{A_{ch1c} := \dfrac{CF_{1c}}{\phi_b \cdot f_y}}$ $\boxed{A_{ch1c} = 7.33\,in^2}$ Case 2c

**Panel A-B/2-4**
Provide Chord Steel as follows: ( For North/South Seismic Acceleration)

Case 2c: 6 Nos. # 11 bars ( A=1.56 $in^2$ ) Provided As = 9.36 in2

=======

### 6.6.3 Check Chord Steel Requirements ( For Cases 1 thru 5 ):

#### 6.6.3.1 For North/South Seismic Acceleration:

$$\boxed{M_{ns1} = \begin{pmatrix} 139859 \\ 53628 \\ 31321 \\ 28051 \\ 23313 \end{pmatrix} ft \cdot kip} \quad \boxed{Depth_1 = \begin{pmatrix} 53 \\ 157 \\ 104 \\ 53 \\ 53 \end{pmatrix} ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

(For diaphragm Moments and Depths values, see Section 6.4.4.2 of this calculation)

Note : 0.90 x Depth1 is taken as lever arm from centroid of compression stress block to centroid of the Chord Reinforcing steel ( See attachment C and Assumption 3.2.3 )

Chord Force (CF): $CF_1 := \dfrac{M_{ns1}}{0.90 \cdot Depth_1}$

$$CF_1 = \begin{pmatrix} 2932 \\ 380 \\ 335 \\ 588 \\ 489 \end{pmatrix} kip \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Required Chord Steel (Ach):

$\phi_b := 0.90$ Capacity Reduction Factor for bending (Ref: 2.2.3, Sect. 9.3.2.1)

$$\boxed{A_{ch1} := \frac{CF_1}{\phi_b \cdot f_y}} \qquad A_{ch1} = \begin{pmatrix} 54.3 \\ 7.03 \\ 6.2 \\ 10.89 \\ 9.05 \end{pmatrix} in^2 \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Provide Chord Steel as follows: ( Ach1 **For North/South Seismic Acceleration)**

| | | | |
|---|---|---|---|
| Case 1: | 38 Nos. | # 11 bars ( A=1.56 $in^2$ ) | Provided As = 59.28 in2 |
| Case 2: | 7 Nos. | # 11 bars ( A=1.56 $in^2$ ) | Provided As = 10.92 in2 |
| Case 3: | 5 Nos. | # 11 bars ( A=1.56 $in^2$ ) | Provided As = 7.80 in2 |
| Case 4: | 8 Nos. | # 11 bars ( A=1.56 $in^2$ ) | Provided As = 12.48 in2 |
| Case 5: | 7 Nos. | # 11 bars ( A=1.56 $in^2$ ) | Provided As = 10.92 in2 |

**6.6.3.2 For East/West Seismic Acceleration:**

$$\boxed{M_{ew1} = \begin{pmatrix} 36503 \\ 232312 \\ 54440 \\ 43135 \\ 37558 \end{pmatrix} ft \cdot kip} \quad \boxed{Depth_2 = \begin{pmatrix} 118 \\ 266 \\ 61 \\ 148 \\ 118 \end{pmatrix} ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

(For diaphragm Moments and Depths values, see Section 6.4.6.1 of this calculation)

Note : 0.90 x Depth2 is taken as lever arm from centroid of compression stress block to centroid of Chord Re-bar steel ( See attachment C and Assumption 3.2.3 )

Chord Force (CF):

$$CF_2 := \frac{M_{ew1}}{0.90 \cdot Depth_2} \qquad CF_2 = \begin{pmatrix} 344 \\ 970 \\ 992 \\ 324 \\ 354 \end{pmatrix} kip \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Required Chord Steel (Ach):

$\phi_b := 0.90$ Capacity Reduction Factor for bending (Ref: 2.2.3, Sect. 9.3.2.1)

$$\boxed{A_{ch2} := \frac{CF_2}{\phi_b \cdot f_y}} \qquad A_{ch2} = \begin{pmatrix} 6.4 \\ 18 \\ 18.4 \\ 6 \\ 6.5 \end{pmatrix} in^2 \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \\ \text{Case 5} \end{matrix}$$

Provide Chord Steel as follows: ( Ach 2 **For East/ West Seismic Acceleration )**

| | | | |
|---|---|---|---|
| Case 1: | 6 Nos. | # 11 bars ( A = 1.56 $in^2$ ) | Provided As = 9.36 in2 |
| Case 2: | 14 Nos. | # 11 bars ( A = 1.56 $in^2$ ) | Provided As = 21.84 in2 |
| Case 3: | 14 Nos. | # 11 bars ( A = 1.56 $in^2$ ) | Provided As = 21.84 in2 |
| Case 4: | 5 Nos. | # 11 bars ( A = 1.56 $in^2$ ) | Provided As = 7.80 in2 |
| Case 5: | 6 Nos. | # 11 bars ( A = 1.56 $in^2$ ) | Provided As = 9.36 in2 |

**6.6.3.3 Use <u>Chord Steel</u> for diaphragm as follows (<u>For N/S & E/W Seismic Acceleration</u>)**

[ Note: See Attachment C for locations of these Chord Reinforcement in the slab panel (Typ) ]

**CHORD STEEL SUMMARY**

| | | | Between |
|---|---|---|---|
| Case 1 (El. 100'): | N/S Loading: | 38 - # 11 bars along column lines A | 4 & 7 |
| | | 38 - # 11 bars along column lines B | 4 & 7 |
| | E/W Loading: | 6 - # 11 bars along column lines 7 | A & B |
| | | 6 - # 11 bars along column lines 4 | A & B |
| Case 2 (El. 80'): | N/S Loading: | 6 - # 11 bars along column lines A | 1 & 4 |
| | | 6 - # 11 bars along column lines B | 1 & 4 |
| | | 7 - # 11 bars along column lines B | 1 & 7 |
| | | 7 - # 11 bars along column lines D | 1 & 7 |
| | E/W Loading: | 14 - # 11 bars along column lines 7 | B & D |
| | | 14 - # 11 bars along column lines 4 | A & B |
| | | 14 - # 11 bars along column lines 1 | A & D |
| Case 3 (El. 40'): | N/S Loading: | 5 - # 11 bars along column lines B | 1 & 2.1 |
| | | 5 - # 11 bars along column lines C | 1 & 2.1 |
| | E/W Loading: | 14 - # 11 bars along column lines 2.1 | B & C |
| | | 14 - # 11 bars along column lines 1 | B & C |
| Case 4 (El. 40'): | N/S Loading: | 8 - # 11 bars along column lines A | 1 & 4 |
| | | 8 - # 11 bars along column lines B | 1 & 4 |
| | | 8 - # 11 bars along column lines C | 1 & 6 |
| | | 8 - # 11 bars along column lines D | 1 & 6 |
| | E/W Loading: | 5 - # 11 bars along column lines 4 | A & B |
| | | 5 - # 11 bars along column lines 6 | C & D |
| | | 5 - # 11 bars along column lines 1 | A & B |
| | | 5 - # 11 bars along column lines 1 | C & D |
| Case 5 (El. 32'): | N/S Loading: | 7 - # 11 bars along column lines A | 4 & 7 |
| | | 7 - # 11 bars along column lines B | 4 & 7 |
| | E/W Loading: | 6 - # 11 bars along column lines 7 | A & B |
| | | 6 - # 11 bars along column lines 4 | A & B |

# 7. RESULTS AND CONCLUSIONS

## 7.1 RESULTS

The results from this calculation are as follows:

**7.1.1** The diaphragms reinforcement as computed (See Section 6.5.6 showing the reinforcement summary).

**7.1.2** The chord reinforcement as computed (See Section 6.6.3 showing the reinforcement summary).

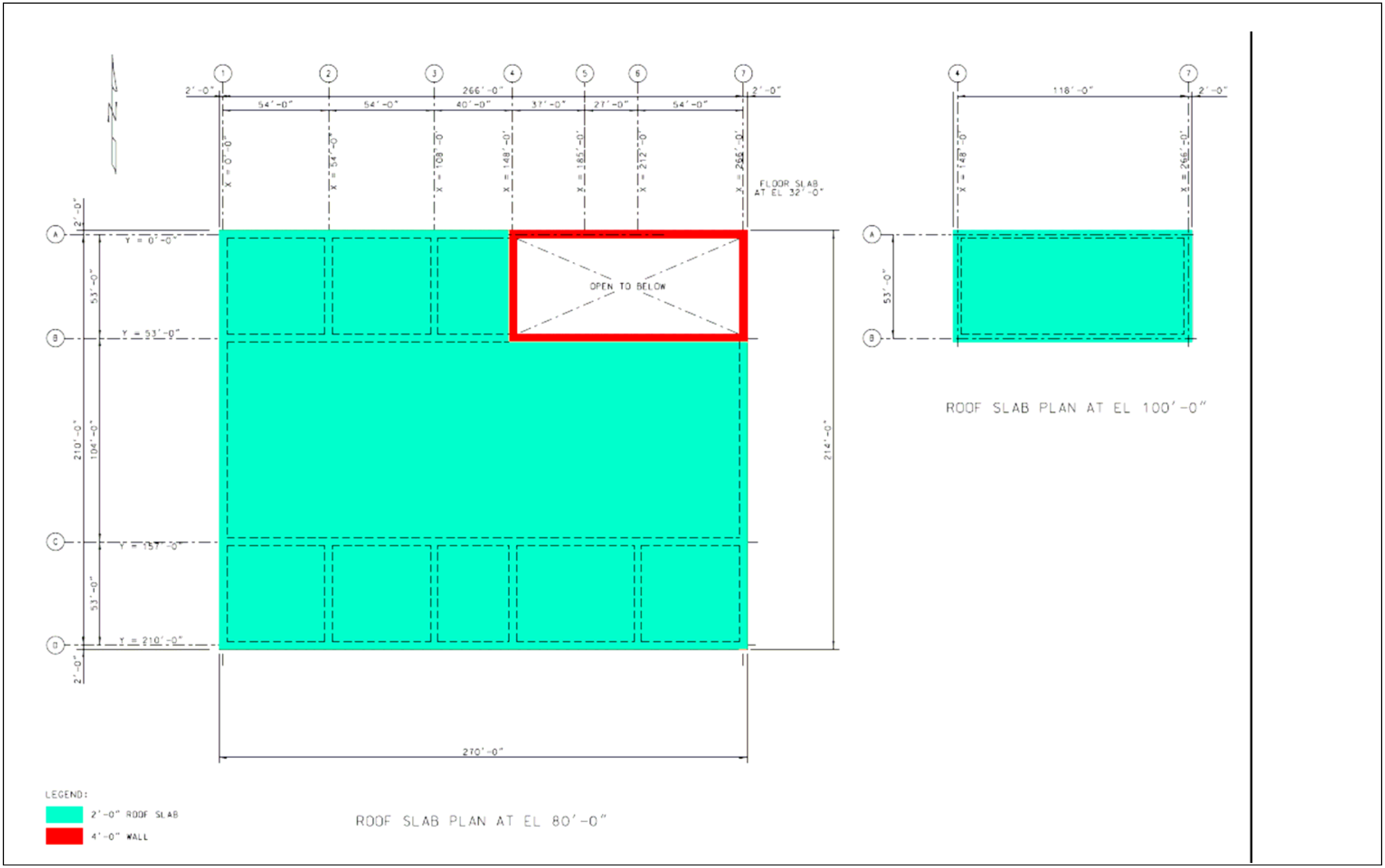
## 7.2 CONCLUSIONS

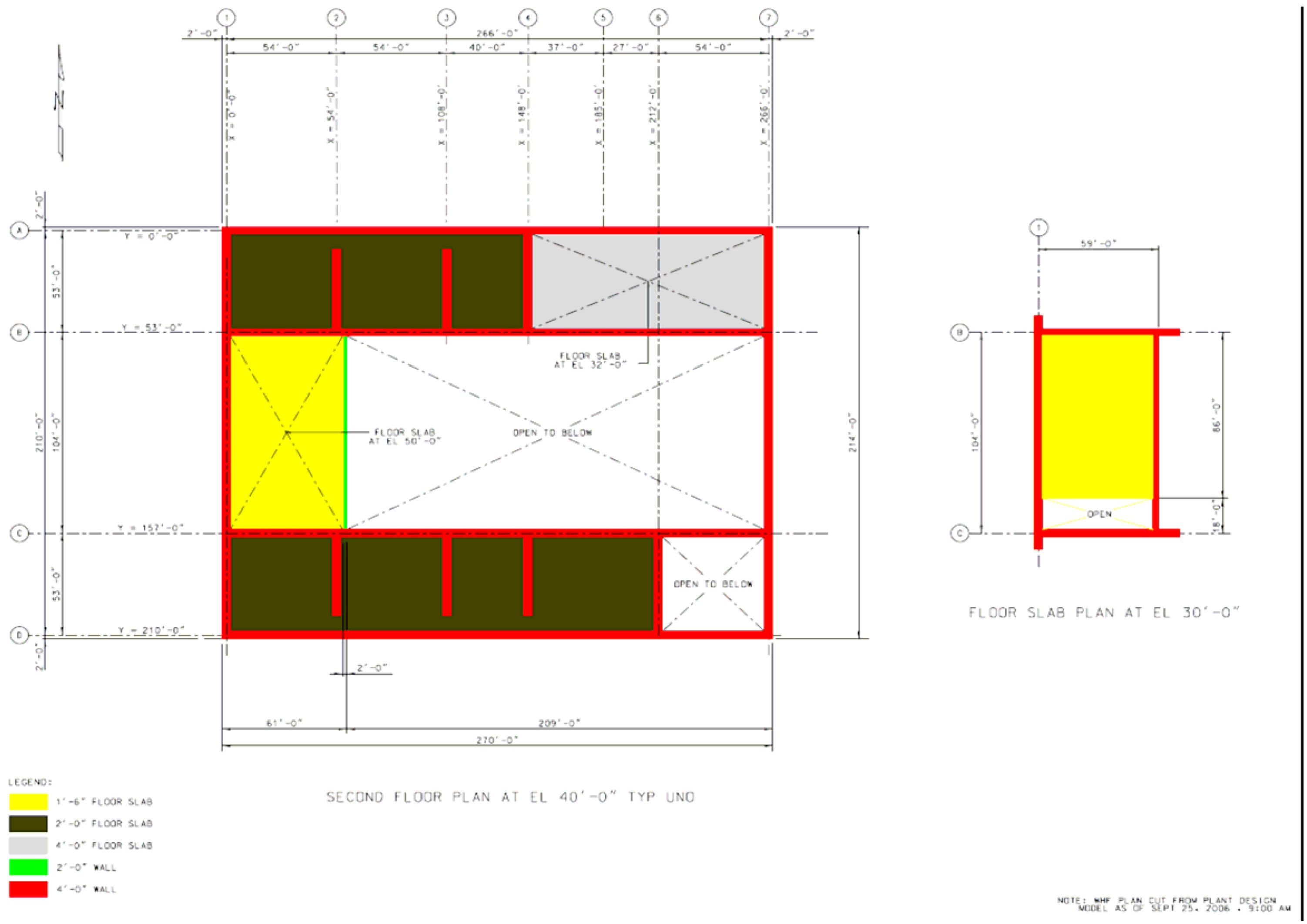
Results from this calculation demonstrate that for the slabs investigated a reasonable slab design is achieved for the imposed design loads. The slab/ diaphragm reinforcement provided in Section 6.5.6 of the calculation is reasonable for the type of structure under consideration and the types of loads applied to this structure. The Reinforcement Ratio (see Section 6.5.6) shows that there is adequate margin for use in consideration of larger seismic events in the probabilistic risk assessment.

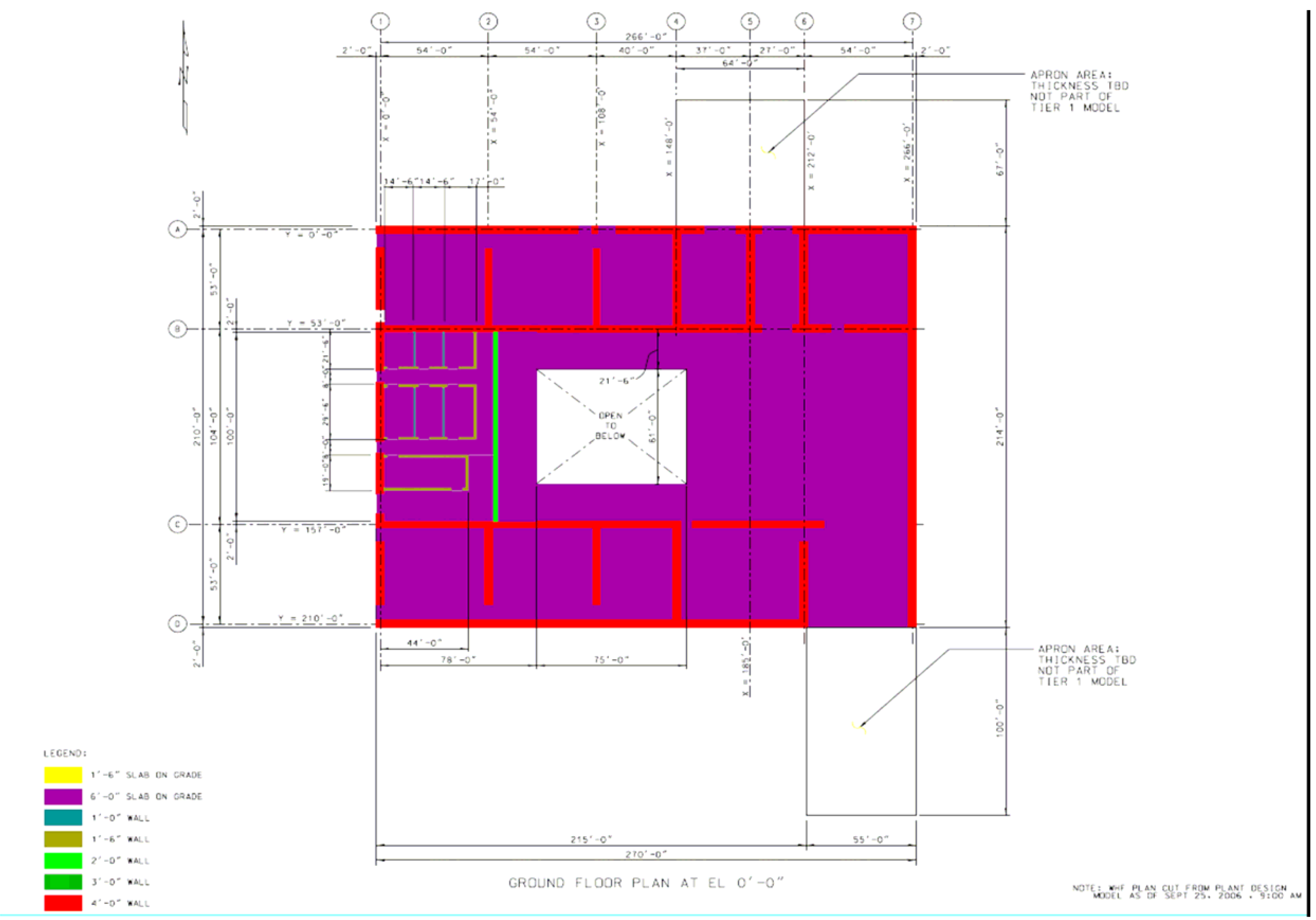
Chord reinforcement provided in Section 6.6.3 is based on conservatively putting the reinforcement as shown in Attachment B so that the lever arm from the centroid of compression stress block to the centroid of chord reinforcement is 0.9 x Depth of diaphragm (see Attachment C). During the detailed design phase of the project three dimensional finite element model will yield reduction in the chord reinforcement required.

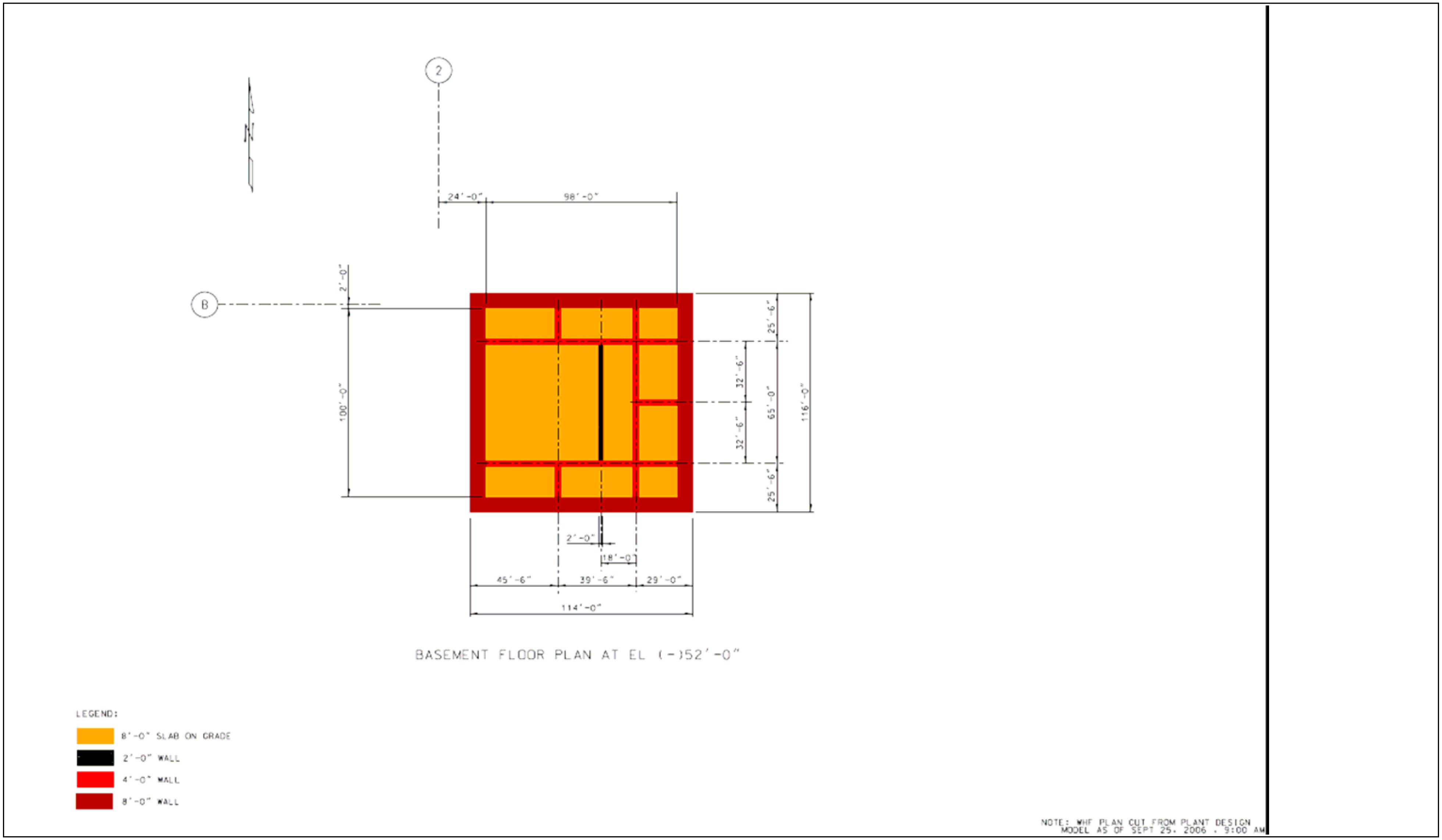
Results from this analysis are preliminary and should be used only in the preliminary design phase of the project. The results of the calculation are adequate for use in the structural design calculations being performed as part of the Tier 1 seismic analysis.

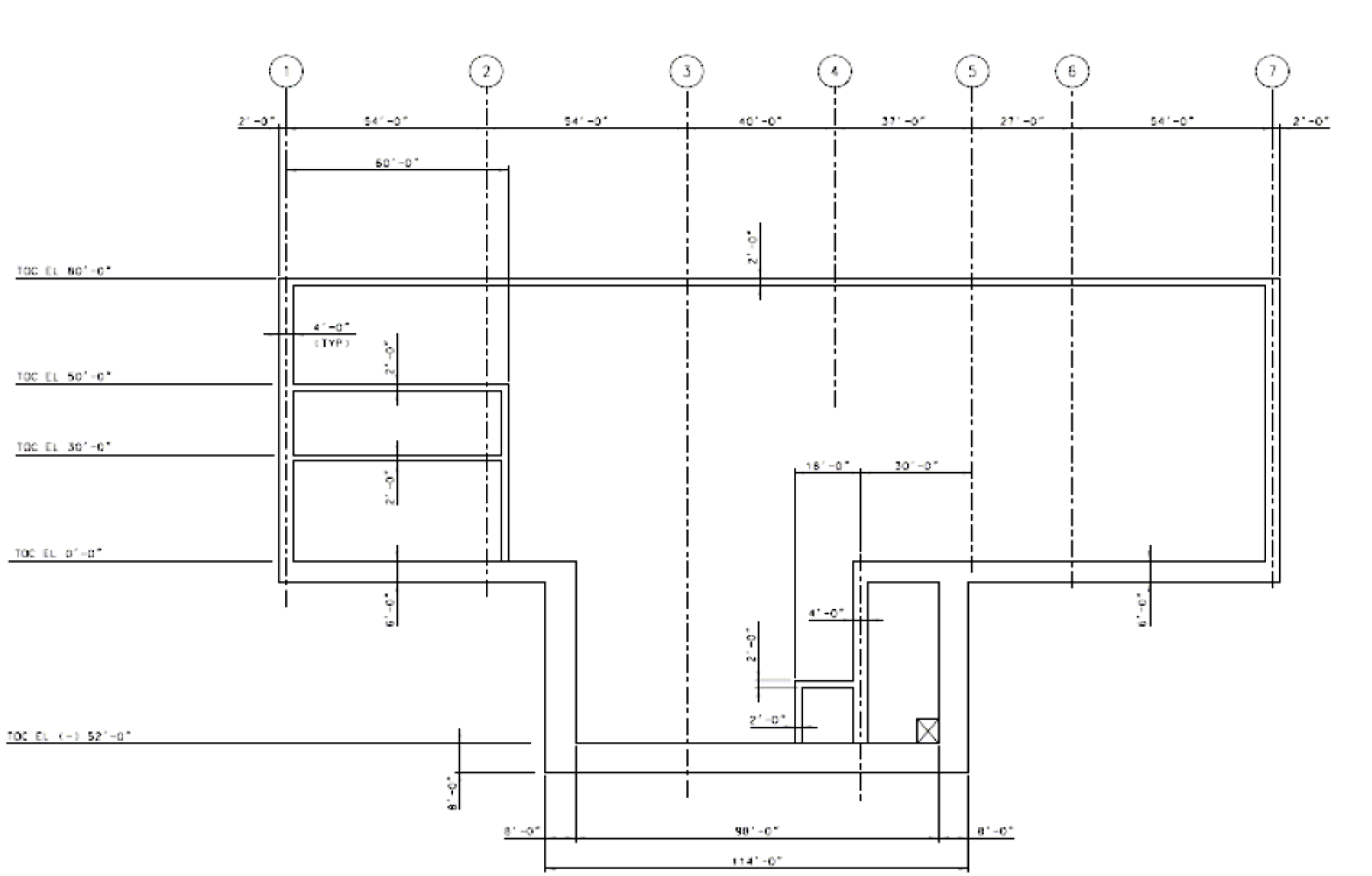
# ATTACHMENT A. WHF - PLAN & SECTION SKETCHES


OPEN TO BELOW
FLOOR SLAB AT EL 32'-0"
266'-0"
270'-0"
214'-0"
210'-0"
118'-0"
LEGEND:
2'-0" ROOF SLAB
4'-0" WALL
ROOF SLAB PLAN AT EL 80'-0"
ROOF SLAB PLAN AT EL 100'-0"

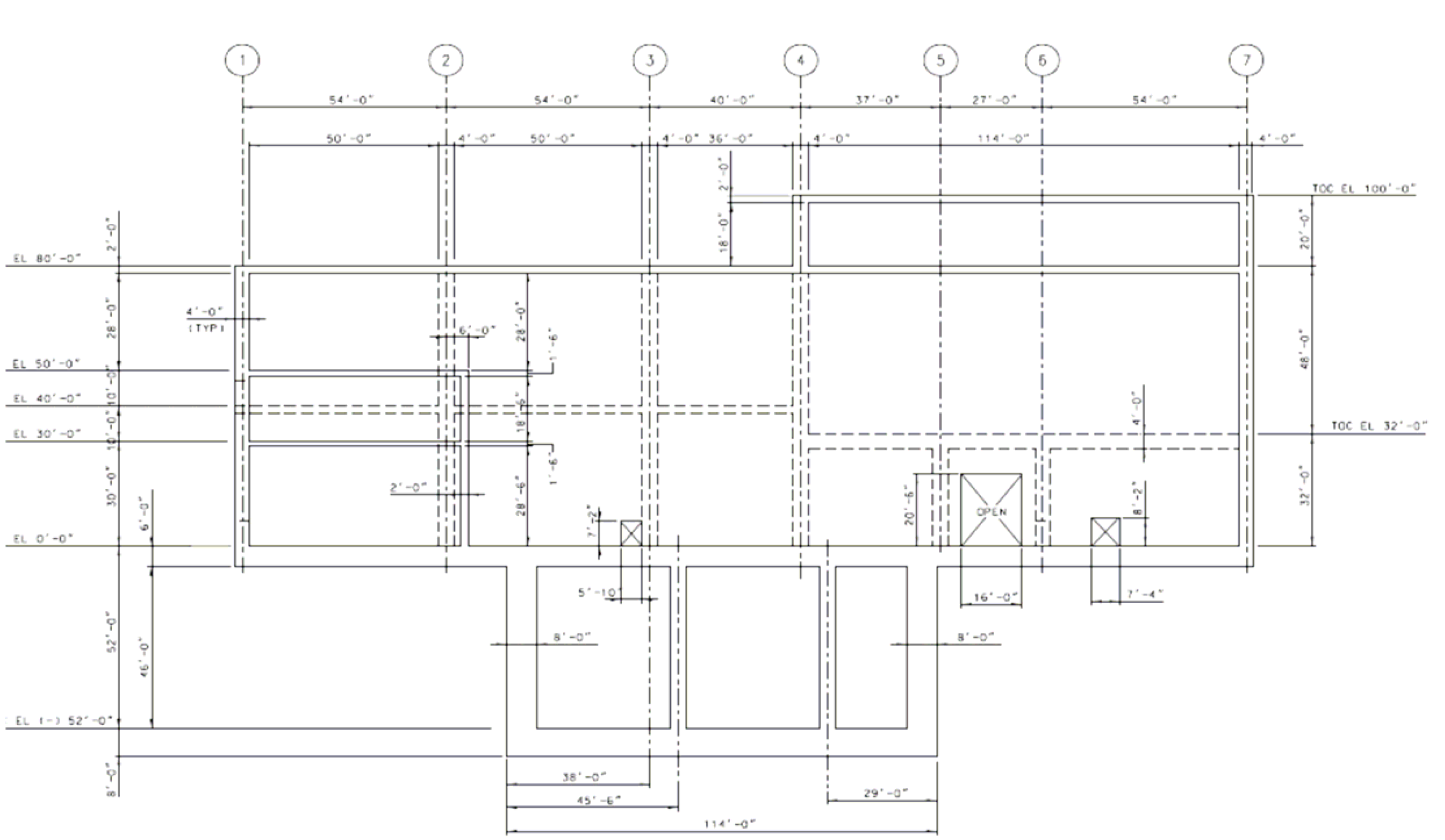

FLOOR SLAB AT EL 32'-0"
FLOOR SLAB AT EL 50'-0"
OPEN TO BELOW
OPEN TO BELOW
SECOND FLOOR PLAN AT EL 40'-0" TYP UNO
OPEN
FLOOR SLAB PLAN AT EL 30'-0"
LEGEND:
1'-6" FLOOR SLAB
2'-0" FLOOR SLAB
4'-0" FLOOR SLAB
2'-0" WALL
4'-0" WALL
NOTE: WHF PLAN CUT FROM PLANT DESIGN MODEL AS OF SEPT 25, 2006, 9:00 AM


APRON AREA:
THICKNESS TBD
NOT PART OF
TIER 1 MODEL
APRON AREA:
THICKNESS TBD
NOT PART OF
TIER 1 MODEL
OPEN TO BELOW
LEGEND:
1'-6" SLAB ON GRADE
6'-0" SLAB ON GRADE
1'-0" WALL
1'-6" WALL
2'-0" WALL
3'-0" WALL
4'-0" WALL
GROUND FLOOR PLAN AT EL 0'-0"
NOTE: WHF PLAN CUT FROM PLANT DESIGN MODEL AS OF SEPT 25, 2006 , 9:00 AM

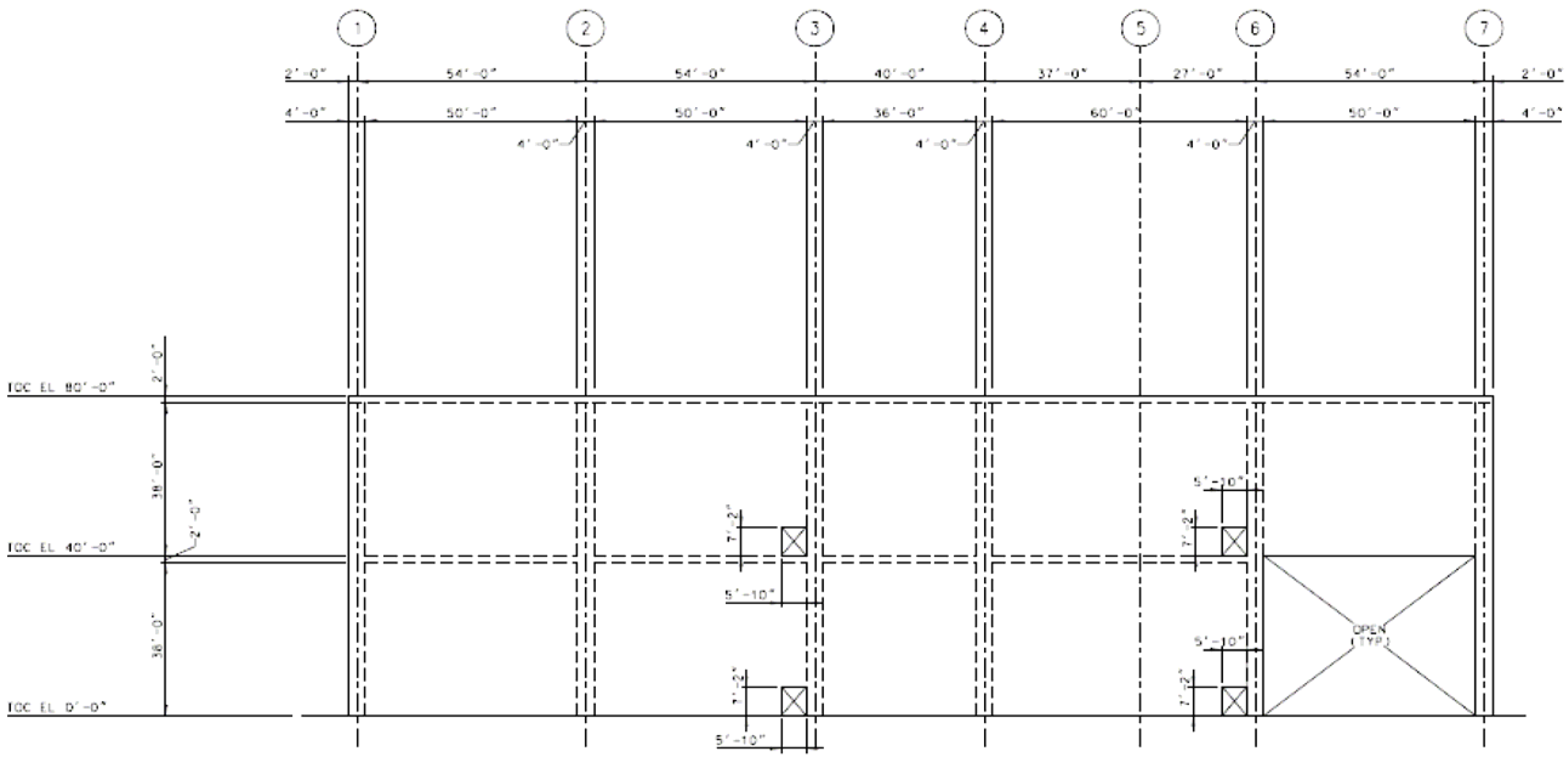

2
B
24'-0"
98'-0"
2'-0"
100'-0"
25'-6"
32'-6"
32'-6"
65'-0"
116'-0"
25'-6"
2'-0"
18'-0"
45'-6"
39'-6"
29'-0"
114'-0"
BASEMENT FLOOR PLAN AT EL (-)52'-0"
LEGEND:
8'-0" SLAB ON GRADE
2'-0" WALL
4'-0" WALL
8'-0" WALL
NOTE: WHF PLAN CUT FROM PLANT DESIGN MODEL AS OF SEPT 25, 2006, 9:00 AM




LONGITUDINAL SECTION BETWEEN COL 'B' & 'C'
LOOKING NORTH

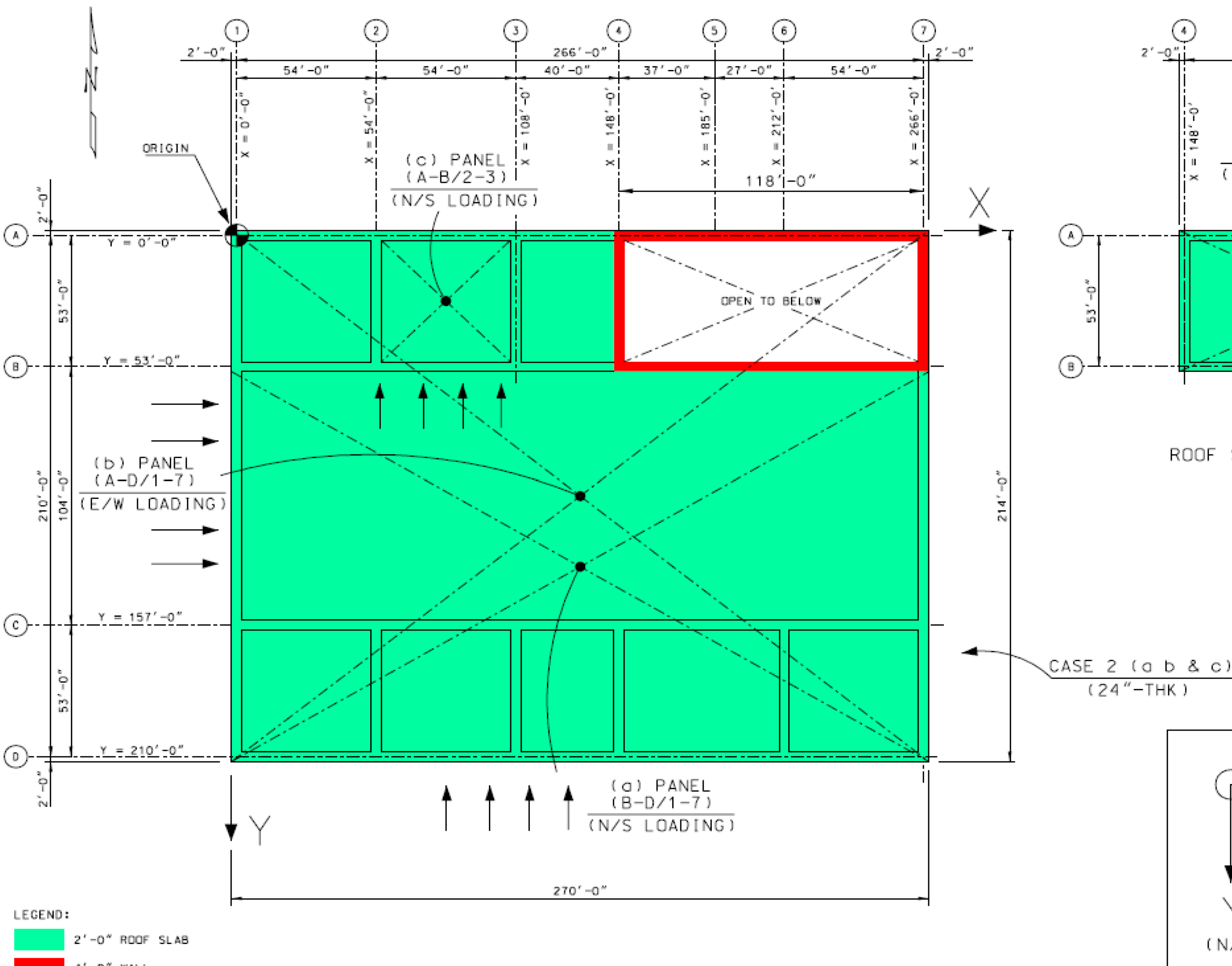


ELEVATION ALONG COL "B"
(LOOKING NORTH)

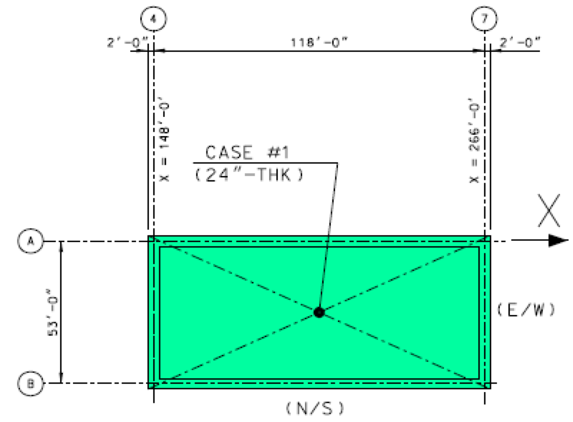



SOUTH WALL-ELEVATION ALONG COL LINE "D"
(LOOKING NORTH)

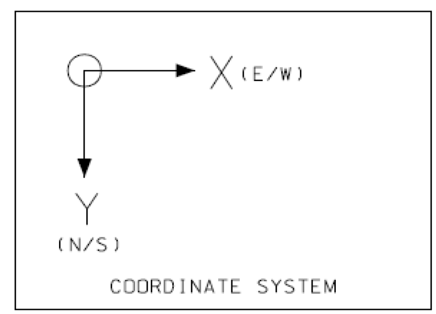
# ATTACHMENT B. WHF – DIAPHRAGM PLANS (SHOWING PANEL DESIGN CASES)

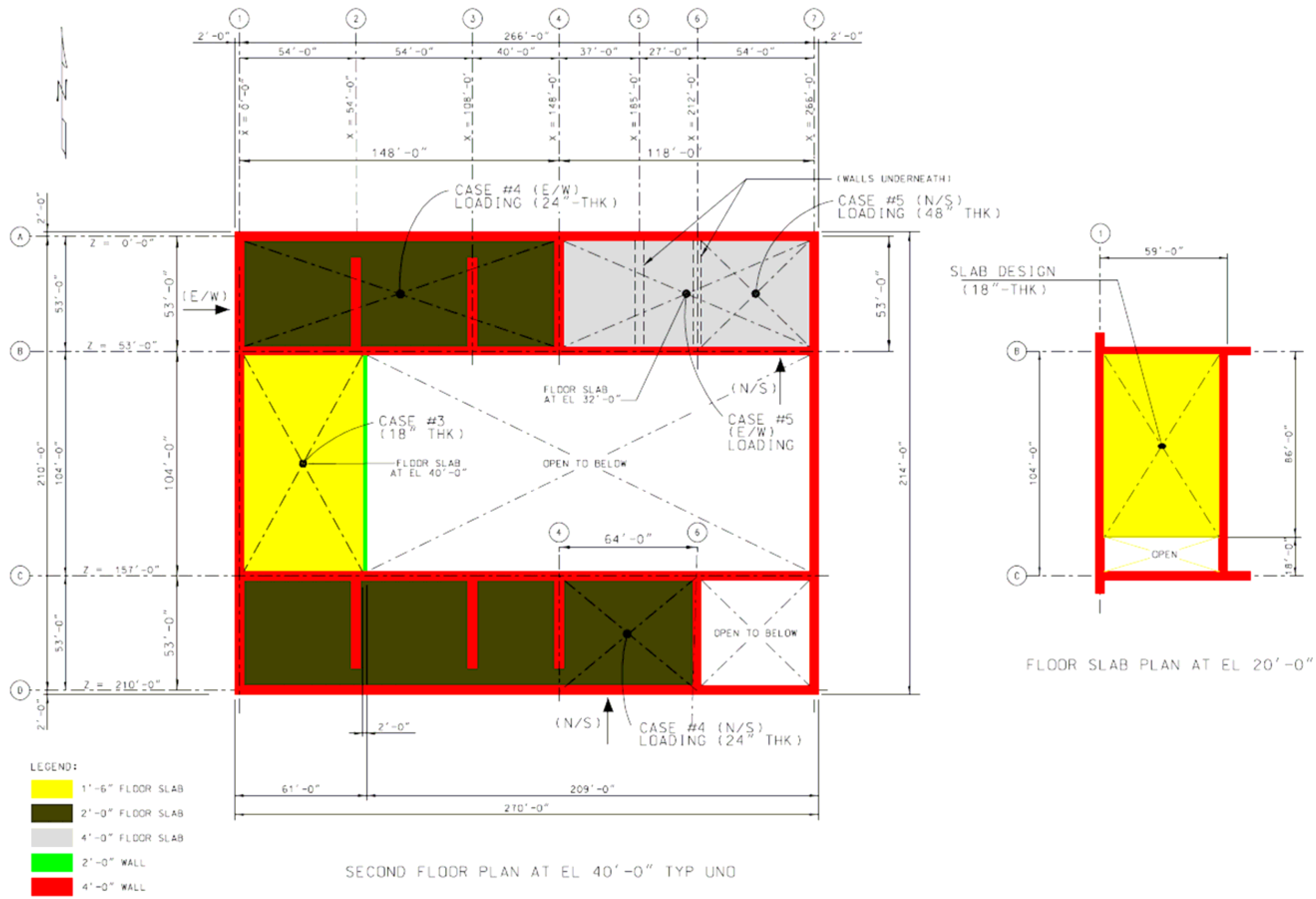



ROOF SLAB PLAN AT EL 80'-0"

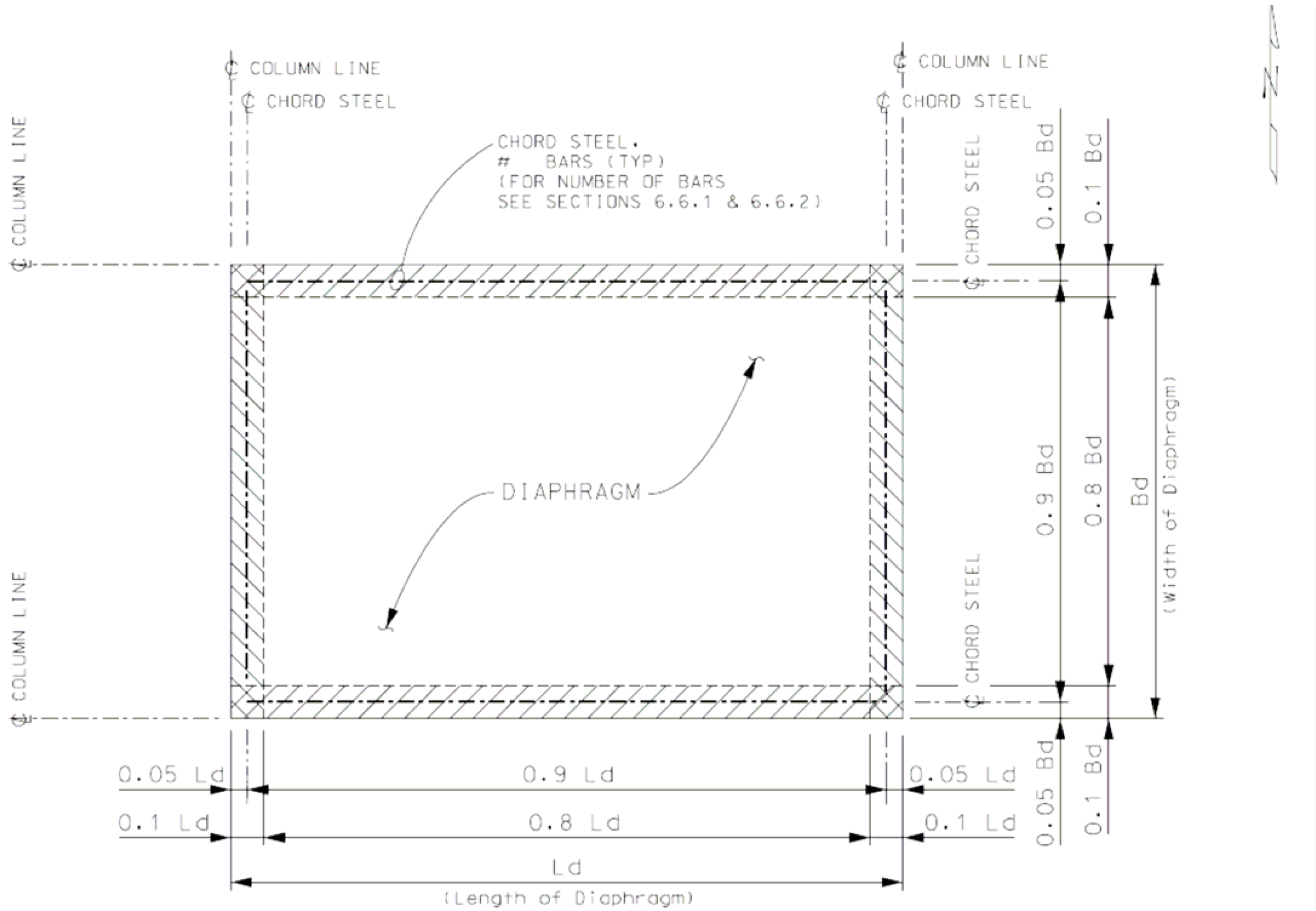



ROOF SLAB PLAN AT EL 100'-0"




266'-0"
54'-0"
54'-0"
40'-0"
37'-0"
27'-0"
54'-0"
2'-0"
148'-0"
118'-0"
CASE #4 (E/W) LOADING (24"-THK)
(WALLS UNDERNEATH)
CASE #5 (N/S) LOADING (48" THK)
(E/W)
Z = 0'-0"
Z = 53'-0"
Z = 157'-0"
Z = 210'-0"
FLOOR SLAB AT EL 32'-0"
(N/S)
CASE #3 (18" THK)
FLOOR SLAB AT EL 40'-0"
OPEN TO BELOW
CASE #5 (E/W) LOADING
64'-0"
CASE #4 (N/S) LOADING (24" THK)
53'-0"
104'-0"
210'-0"
214'-0"
61'-0"
209'-0"
270'-0"
LEGEND:
1'-6" FLOOR SLAB
2'-0" FLOOR SLAB
4'-0" FLOOR SLAB
2'-0" WALL
4'-0" WALL
SECOND FLOOR PLAN AT EL 40'-0" TYP UNO
SLAB DESIGN (18"-THK)
59'-0"
86'-0"
18'-0"
OPEN
FLOOR SLAB PLAN AT EL 20'-0"

# ATTACHMENT C. WHF – DIAPHRAGM PANEL SKETCH (SHOWING CHORD




[PLAN SHOWING TYPICAL CHORD STEEL FOR DIAPHRAGM]
(NTS)

LEGEND: = LOCATION OF CHORD STEEL FOR N/S SEISMIC ACCELERATION
= LOCATION OF CHORD STEEL FOR E/W SEISMIC ACCELERATION